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Vlad G. Perlea
U.S. Army Corps of Engineers, Sacramento, CA

Michael H. Beaty
Beaty Engineering LLC, Beaverton, OR

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Vlad G. Perlea
US Army Corps of Engineers, Sacramento District
Sacramento, California-USA 95814

Michael H. Beaty
Beaty Engineering LLC
Beaverton, Oregon-USA 97007

ABSTRACT

Guidance for the Corps of Engineers requires the use of seismic deformation analysis in the evaluation of existing dams for seismic loads and in the validation of remediation design for seismically deficient embankment dams. The Corps uses a phased approach for evaluating the seismic safety. For dams with significant seismic loads, problem soils, high risk, or those where simple analyses identified seismic concerns, a more advanced deformation analysis is generally required. A major challenge in these analyses is the selection of a reliable constitutive model when liquefiable materials are present. The computer program FLAC, in conjunction with a modified version of the UBCSAND constitutive model, has been instrumental in determining the necessity of seismic retrofit at Corps' embankment dams and in selecting effective remediation alternatives. The primary objective of this paper is to describe the current practice of the Corps in applying a phased analysis approach and to discuss selected considerations in performing these analyses.

INTRODUCTION

In two memoranda to the Corps of Engineers dated April 14 and May 20, 1953, R.W. Whitman and D.W. Taylor first proposed an important concept: the effects of earthquakes on embankment stability should be assessed in terms of the deformations they produce rather than on a factor of safety against slope failure. Nathan Newmark, who served on an advisory board with D.W. Taylor for the Corps of Engineers, further developed Whitman's concept and presented a proposed method of analysis in his Rankine Lecture of 1965 (Marcuson III et al. 2007).

In the aftermath of the near failure of the Lower San Fernando Dam in 1971, the US Army Corps of Engineers decided to re-evaluate all dams under its jurisdiction for seismic stability. Many of these dams were designed or constructed in the 1950's when the current knowledge and capabilities in seismic design were not available. The highest priority in this effort was given to dams built of hydraulic fill and to those on recent alluvium foundation.

Since these early developments, seismic safety evaluations have been recognized as a periodic need for all dams of consequence. Several factors contribute to this need, including developments in our understanding of potential seismic loading and soil response, improvements in the available tools to model embankment response and deformations, and changes in embankment operation or performance during its life.

A wide range of tools are available for estimating the response of embankments to seismic loading. It is generally assumed that simplified methods that are properly formulated and applied tend to provide conservative results. This is not always the case, and simplified methods should always be applied with great care and experienced judgment. Increasing the sophistication of the analysis has the potential for reducing unnecessary conservatisms. Advanced analysis may reduce the final cost of any remediation measures by improving the understanding of potential embankment response. The evaluation of the seismic stability in progressive stages is aimed at reducing the need for sophisticated and costly analyses if an adequate margin of safety can be confirmed through simple and efficient procedures.

In what follows, the practices of the Corps of Engineers for various levels of seismic deformation evaluation are described. The practices are divided into two groups: simplified procedures and advanced procedures. Selected examples are provided with particular attention given to two recent projects where the authors had a major role in analyses: Tuttle Creek Dam in Kansas and Success Dam in California. Although the results from several analysis approaches are presented for both dams, the results are not always directly comparable due to changes in the site characterization, seismic loading, or other parameters that may have occurred between the analysis phases.

Tuttle Creek Dam is a rolled earth fill and hydraulic fill embankment, 2,285 m (7,500 feet) in length, standing 41.8 m (137 feet) high, with a crown width of 15.2 m (50 feet) and a base width of 320 m (1,050 feet). The dam is located upstream of Manhattan, Kansas. The typical geometry with embankment and foundation zones is shown in Fig. 1.

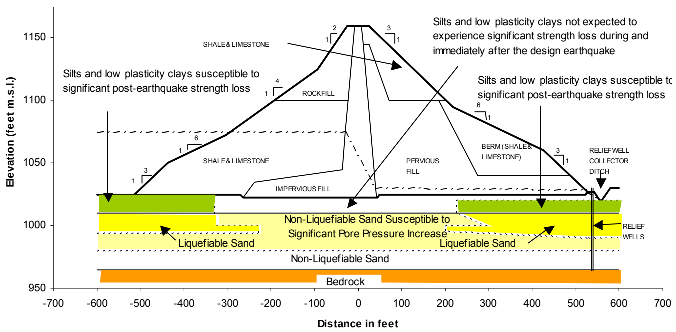


Fig. 1. Embankment and foundation zones for Tuttle Creek Dam – distorted scale (Perlea et al. 2004).

Seismic and geotechnical investigations established that a strong earthquake generated from a nearby active fault zone could induce liquefaction of the alluvial foundation soil under the lower portions of both the upstream and downstream slopes (Perlea et al. 2004). The evaluations and deformation analyses led to an embankment remediation that includes a series of transverse shear walls near the downstream toe constructed of concrete slurry. The construction of the remediation was completed in September 2009.

Success Dam is a zoned earth-filled embankment located upstream of Porterville, California. It has a crest length of 1,064 m (3,490 feet) and a maximum height of 44.2 m (145 feet). The typical geometry with embankment and foundation zones is shown in Fig. 2.

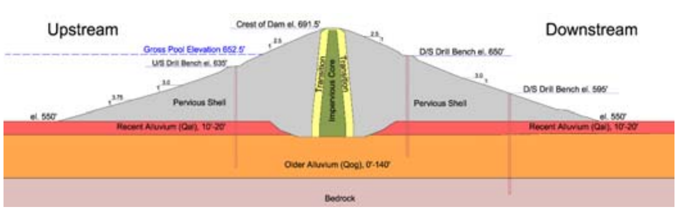


Fig. 2. Typical cross section of Success Dam, California.

The upper layer of foundation, consisting of recent alluvium, was found to be liquefiable in addition to some locations in the upstream shell. Beneath the recent alluvium are zones of older, non-liquefiable alluvium founded on weathered bedrock. The evaluations and deformation analyses have led to a proposed embankment remediation that consists of a buttress, including a replacement core and new shell zones, downstream of the existing embankment.

SIMPLIFIED PROCEDURES

Simplified methods for evaluating seismic deformation may be classified into four broad categories. These categories are briefly summarized and then presented in more detail in the following sections.

<u>Category 1:</u> Screening tests based on case histories or developed using empirical methods.

<u>Category 2:</u> Methods that assume most of the displacements occur during seismic shaking and are due to the action of horizontal inertia forces induced by the earthquake. These methods assume the post-earthquake movements are negligible. They are primarily applicable to cases where there is no significant loss of material strength during the earthquake.

<u>Category 3:</u> Methods that assume movements are primarily caused by gravitational forces acting on an embankment following the seismic reduction of material strength. Displacements occur as the embankment deforms to achieve static equilibrium. These methods are generally used when liquefiable or sensitive materials are present.

<u>Category 4:</u> Methods that combine the assumptions in categories (2) and (3), such as those that assume residual strengths are mobilized early in the earthquake.

<u>Category 1 – Screening and Empirical Evaluations</u>

<u>Screening Test:</u> There have been numerous embankment dams subjected to moderate seismic loading that have experienced minor to no damage. Experience has shown that a well built dam on a good foundation is not likely to be damaged in a moderate earthquake. Seed et al. (1978) recommended screening criteria that can be used to avoid spending undue attention and resources on dams that are unlikely to suffer significant damage during an earthquake. These original criteria have been somewhat revised by the Corps and are summarized below. These criteria are currently being reviewed and evaluated. Deformation analyses are not typically required in Corps practice for low to moderate height dams (< 60 m / 200 feet high) if all of the following eight conditions are satisfied.

- Dam and foundation materials are dense, not subject to liquefaction, and do not include sensitive clays. Foundations containing substantial deposits of recent alluvium are a potential concern.

- The dam is well built and densely compacted to at least 95% of the laboratory maximum dry density, or to a relative density greater than 80%.

- The slopes of the dam are 3:1 (H:V) or flatter, and/or the phreatic line is well below the downstream face of the embankment.

- The predicted peak horizontal ground acceleration (PGA) at the base of the embankment is no more than 0.20g. Compacted clay embankments on rock or stiff clay foundations may offer additional resistance to deformations. Somewhat higher allowable PGA values may be justifiable for these dams on a case-by-case basis, although the PGA criterion should not exceed 0.35g (USBR, 1989).

- The static factors of safety for all potential failure surfaces involving loss of crest elevation (other than shallow surficial slides) are greater than 1.5 under the loading and pore-pressure conditions expected immediately prior to the earthquake;

- The freeboard at the time of the earthquake is at least 3 to 5 percent of the dam height plus alluvial foundation, and not less than 0.9 m (3 feet). Special attention should be given to the presence and suitability of filters for dams with modest freeboard.

- There are no appurtenant features related to the safety of the dam that would be harmed by small movements of the embankment.

- There have been no historic incidents at the dam that may indicate a limitation in its ability to survive an earthquake.

Special conditions may warrant further study, such as dams susceptible to internal erosion but without filters or the presence of active faults within the foundation. Dams having significant consequences for failure may require adjusted minimum criteria, such as an increase in the required freeboard.

Empirical Methods – General: Empirical methods correlate observations of dam performance following an earthquake event to selected criteria describing the dam or earthquake loading. These methods are primarily useful for evaluating the anticipated dam response to a deterministically-derived earthquake. Two simple charts for performing an empirically-based evaluation are described below.

Empirical Relation for Damage: A listing of historic dam performance during earthquakes was made available by the United States Society on Dams (USSD 2003, Appendix A). This list includes a summary of seismic loading and damage rating for over 300 dams. Case histories of 160 embankment dams, including 12 hydraulic fill dams, were selected by the authors from this list and used to prepare the graph of damage versus event description shown in Fig. 3.

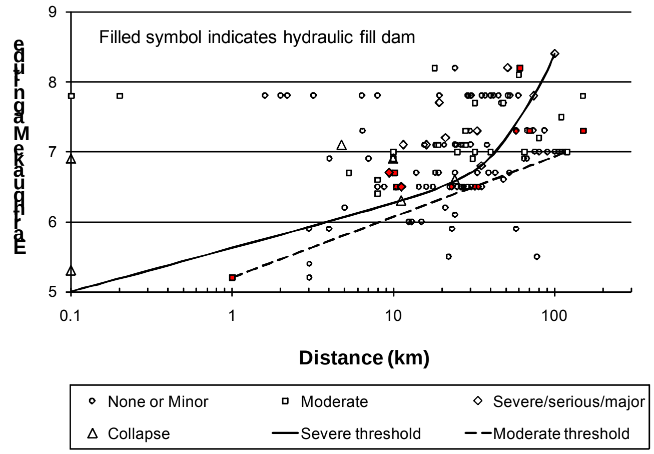


Fig. 3. Severity of damage to earthfill embankment dams based on case histories (1906 – 2001).

The damage rating is grouped into three categories: none or minor, moderate, and severe. The earthquake loading is represented by a combination of magnitude and distance to fault (either epicentral or closest distance depending on the case history). Fig. 3 does not differentiate between the various dam heights, slopes, foundation conditions, or general quality. The threshold curves indicated on the figure are considered conservative trends defining the boundaries between the observed categories of damage.

The USSD document does not specify the type of magnitude measure used to describe each earthquake event. Because the writers' intent was to establish a threshold magnitude, the minimum value from the USSD list and the moment magnitude published by USGS, California Geological Survey or COSMOS website was considered in building the graph in Fig. 3. For example it was considered M = 8.4 for the 1964 Good Friday, AK earthquake, as in the USSD list, although USGS gives M_w = 9.24; but M_w = 7.3 for the 1952 Kern County, CA earthquake as specified by USGS, instead of M = 7.7 per USSD.

Several interesting features were identified from the underlying data. For example, the "severe damage" threshold was not defined by the case histories for hydraulic fill dams. There were also no reported cases of damaged dams at distances in excess of 150 km. It also appears that $M_w < 5$ earthquakes are unlikely to damage most embankment dams even if they occur near the dam. However, many dam locations have the potential for background seismicity in excess of M_w 5.

Empirical Relation for Crest Settlement (no liquefaction): Swaisgood (2003) compiled a database of dam response versus earthquake loading and various dam descriptors. Swaisgood developed an empirical relationship, shown in Fig. 4, that relates seismically-induced crest settlement to the height of the dam and alluvial foundation (DH+AT), the earthquake magnitude, and the peak ground acceleration.

This graph may be useful for predicting the likely range of settlements a dam may experience provided there is no liquefaction, or for a check of the reasonableness of predicted settlements. It is interesting to note that the damages were classified as "moderate" when the settlements were greater than 0.1% of (DH+AT) and "serious" if they were in excess of 0.5% of the total height. The plot also shows that the normalized settlement generally did not exceed 1% without the occurrence of liquefaction or a PGA in excess of 0.5 g.

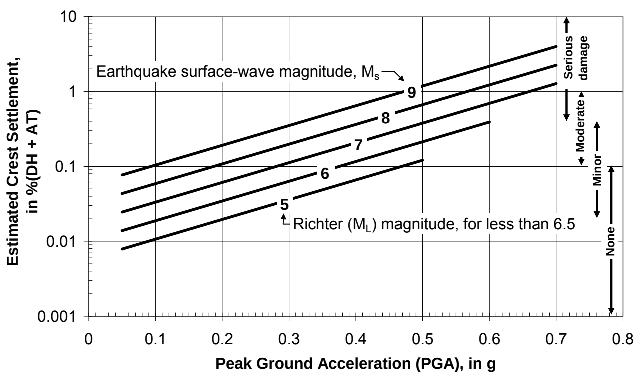


Fig. 4. Empirical correlation for crest settlement due to earthquakes (excluding cases of known liquefaction (adapted from Swaisgood, 2003).

Category 2 – Newmark-Type Approaches

This method is based on the concept that the shear stresses induced during the earthquake may momentarily exceed the available shear strength along the base of a slide mass. The available strength can be expressed as a yield acceleration k_y which is that acceleration that causes yielding on the slide plane when applied uniformly to the slide mass. The applied loading is expressed as the average acceleration of the slide mass assuming there is no yielding on the slide plane (i.e., a decoupled analysis). This approach was first presented by Prof. Newmark (1965).

There is a range in available tools for making Newmark-type estimates of displacements. Some of these tools are aimed at simplifying the analysis procedure, while others attempt to address limitations of the original Newmark approach.

Newmark Integration Analysis: Newmark's method is based on a number of simplifying assumptions: (1) the existence of a well-defined slip surface, (2) a rigid, perfect plastic slide material, (3) permanent strains occur only if the dynamic stress exceeds the shear resistance, and (4) the displacements are presumed to occur in the downslope direction only, thus implying infinite dynamic shear resistance in the upslope direction. The method usually assumes there is negligible loss of shear strength during shaking, although this can be approximately considered by making the yield acceleration a function of time or earthquake-induced displacement.

The most important factor in a traditional Newmark analysis is the selection of the design accelerograms for modeling the seismic motions of the rigid block. The effect of the elastic response of the embankment on the acceleration of the slide mass is not taken into account so the response of the structure is modeled only by k_y. This simplifying assumption causes the Newmark approach to be most appropriate for stiff structures whose response can be approximately represented by an appropriate outcrop acceleration record.

The yield acceleration can be readily determined using conventional limit equilibrium methods by calculating the inertial forces required to lower the factor of safety against block sliding to 1.0. It is typical to evaluate several failure surfaces in addition to that which produces the lowest static factor of safety. The permanent displacement is calculated by double integration of those portions of the accelerogram that exceed the yield acceleration for the selected failure surface. This procedure is illustrated in Fig. 5. No displacements occur until time t_1 when the induced acceleration reaches the yield acceleration for the first cycle, k_{y1}. The relative velocity between the slide mass and underlying material will increase until time t_2 when the acceleration drops below the yield value. The variation in relative velocity is computed by

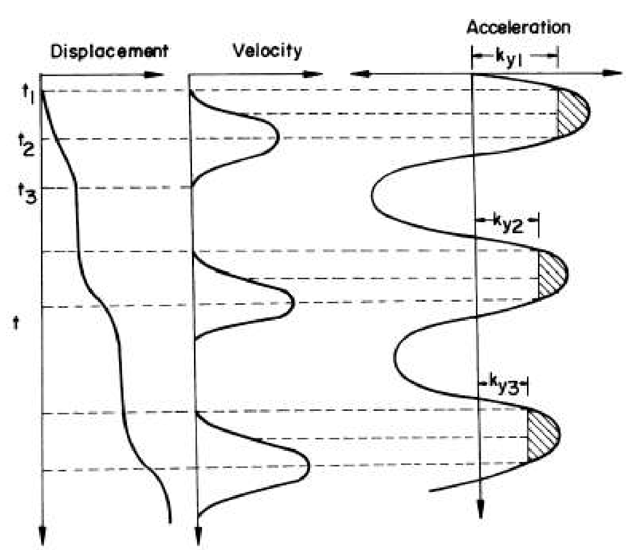


Fig. 5. Integration of accelerograms to determine downslope displacements (after Goodman and Seed, 1966).

integration of the acceleration history over the shaded area. The relative velocity reduces to zero between time t_2 and t_3 since the mobilized strength on the slide plane exceeds the stress being induced by the earthquake motion. Subsequent relative velocity pulses are estimated in a like manner, and the displacement history is computed by integration of the relative velocity versus time relationship.

The integration is usually performed twice for each earthquake record to account for the possibility that the motion could occur in one of two directions. One analysis is performed with the motion in its positive sense, and a second analysis for the motion in its negative sense. This is done because the Newmark analysis generally uses only one side of the acceleration trace, and the two orientations for the same earthquake record can produce significantly different values of Newmark displacement.

The displacements predicted by a 1-D Newmark analysis may be adjusted for effects of embankment properties and geometry using the procedure developed by Hynes-Griffin and Franklin (1984). The predicted deformation results are multiplied by the coefficient α as shown in Fig. 6, where α is derived from the embankment properties and detailed results from the limit-equilibrium analyses. For most practical problems the coefficient α differs from unity by less than 15%.

<u>*Displacement Trends Based on Newmark:*</u> In 1977, Franklin and Chang performed Newmark evaluations of 354 acceleration histories over a range of yield accelerations. The results were summarized as a trend of predicted displacement versus normalized coefficient N/A, where N is the yield acceleration k_y and A is the peak value of the earthquake acceleration. The peak value of acceleration should be obtained from an estimate of the peak bedrock or peak ground acceleration multiplied by an amplification factor that accounts for the quasi-elastic response of the embankment. The results of Franklin and Chang are summarized in Fig. 7, which shows the variation of computed displacements versus N/A.

These calculations were performed by Franklin and Chang to reduce the effort involved in performing the Newmark analysis. Modern computer programs have made the process of performing these analyses much simpler. For example, the USGS has developed a program for performing these analyses that includes a database of 2160 earthquake records from 29 earthquake events (Jibson and Jibson, 2003). The programs make it easier to perform Newmark analyses using earthquake records specifically chosen for the project in question. A recent compilation of Newmark analyses using this database and relating Newmark displacement to Arias Intensity for a yield acceleration of 0.15g is shown in Fig. 8 (Howard, 2009).

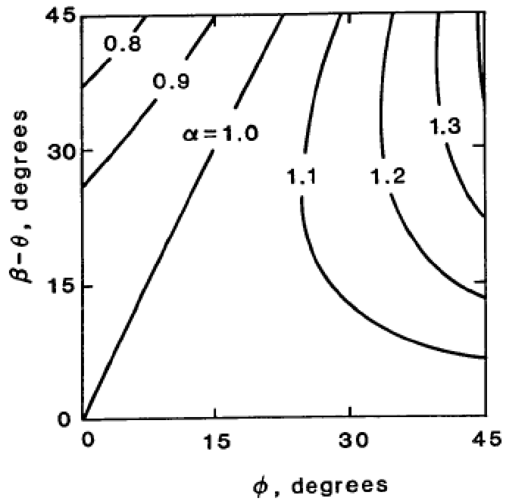


Fig. 6. Values of the coefficient α as a function of the friction angle developed along the failure surface, ϕ, and the difference between the inclination of the resultant of shear stresses β and the critical inclination of the inertia force θ (Hynes-Griffin and Franklin, 1984).

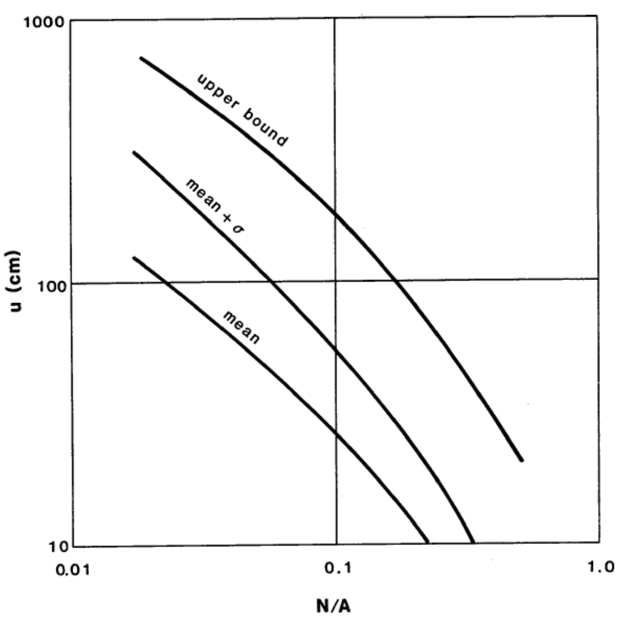


Fig. 7. Permanent displacement u versus N/A, based on 354 accelerograms (Hynes-Griffin and Franklin 1984).

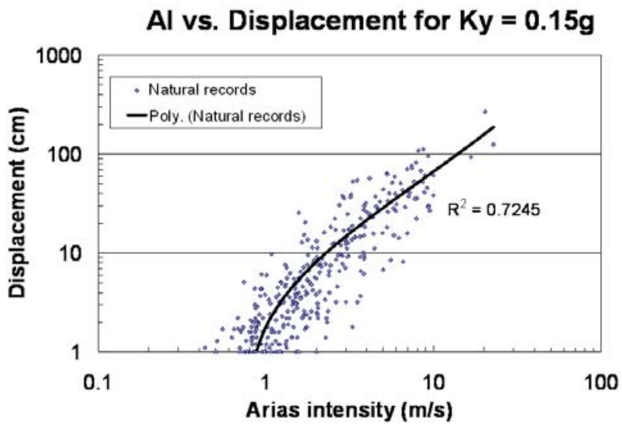


Fig. 8. Displacement versus Arias Intensity using 2160 earthquake records and k_y = 0.15g. Trend line based on computed displacements > 1 cm (after Howard, 2009).

Watson-Lamprey and Abrahamson (2006) proposed an empirical relationship for Newmark displacement that was derived using 6158 recordings. Each record was scaled using 7 different factors and evaluated for 3 different values of k_y, resulting in a total of 129318 analyses. The resulting equation for Newmark Displacement is given in Eq. 1, where D is Newmark displacement in cm, $Sa_{T=1s}$ is the spectral acceleration of the design ground motion at a period of 1 second and 5% damping in g, A_{RMS} is the root mean square of the acceleration in g, PGA is the peak ground acceleration in g, and Dur_{ky} is equal to the total time that the acceleration exceeds the yield acceleration in the direction of maximum displacement. Relationships for A_{RMS} and Dur_{ky} are provided in Eq. 1A and 1B.

$$\begin{aligned}\ln(\mathrm{D(cm)}) &= 5.470 + 0.451 \cdot (\ln(Sa_{T=1s}) - 0.45) \\ &+ 0.0186 \cdot (\ln(Sa_{T=1s}) - 0.45)^2 + 0.596 \cdot (\ln(A_{RMS}) - 1.0) \\ &+ 0.656 \cdot \ln(Sa_{T=1s}/PGA) - 0.0716 \cdot \left(\ln(Sa_{T=1s}/PGA)\right)^2 \\ &+ 0.802 \cdot (\ln(\mathrm{Dur_{ky}}) - 0.74) + 0.0763 \cdot (\ln(\mathrm{Dur_{ky}}) - 0.74)^2 \\ &- \frac{1.0}{0.581 \cdot (\ln(PGA/k_y) + 0.193)}\end{aligned} \tag{1}$$

$$\ln(\mathrm{A_{RMS}(g)}) = -1.167 + 1.02 \cdot \ln(\mathrm{PGA}) \tag{1A}$$

$$\begin{aligned}\ln(\mathrm{Dur_{ky}(s)}) &= -2.775 + 0.956 \cdot \ln(PGA/k_y) \\ &- \frac{1.554}{\ln(PGA/k_y) + 0.39} - 0.597 \cdot \ln(PGA) \\ &+ 0.381 \cdot \ln(Sa_{T=1s}) + 0.334 \cdot M\end{aligned} \tag{1B}$$

Bray and Travasarou (2007): A simplified procedure was proposed by Bray and Travasarou for estimating earthquake-induced permanent displacements in earth dams using a Newmark-type model. This procedure is based on the results from a set of simplified nonlinear analysis using 688 recorded ground motions from 41 earthquakes. The flexibility of the dam system, and the interaction between yielding and seismic loading, were considered by using a nonlinear coupled stick-slip deformable sliding model (Rathje and Bray, 2000). The flexibility of the dam structure is captured through an estimate of the initial fundamental period T_s.

Key parameters of this procedure are the yield acceleration k_y (in g), the initial fundamental period of the embankment, T_s, and the value of spectral acceleration for a damping of 5% and a degraded response period equal to $1.5T_s$. The recommended relationship for estimating the amount of non-zero seismic displacement, D, is given in Eq. 2.

$$\begin{aligned}\ln(\mathrm{D(cm)}) &= -1.10 - 2.83 \cdot \ln(k_y) - 0.333 \cdot \left(\ln(k_y)\right)^2 \\ &+ 0.566 \cdot \ln(k_y) \cdot \ln\left(S_a(1.5 \cdot T_s)\right) \\ &+ 3.04 \cdot \ln\left(S_a(1.5 \cdot T_s)\right) - 0.244 \cdot \left(\ln\left(S_a(1.5 \cdot T_s)\right)\right)^2 \\ &+ 1.5 \cdot T_s + 0.278 \cdot (M - 7) \pm \varepsilon\end{aligned} \tag{2}$$

M in Eq. 2 is the moment magnitude, and ε is a normally distributed random variable with zero mean and a standard deviation σ = 0.66. The yield coefficient is assumed to be constant so the procedure is not appropriate when liquefiable or strain-softening materials are present. The initial coefficients in Eq. 1 (-1.10) should be changed to a value of -0.22 for stiff structures with T_s < 0.05s.

Makdisi and Seed Approach: This method extends the simple 1-dimensional model of a Newmark analysis to consider the 2-dimensional dynamic response of a typical embankment (Makdisi and Seed, 1978). Two-dimensional equivalent linear analyses were performed on several embankment sections using a small group of earthquake records. Section heights of 23, 41, and 46 m (75, 135, and 150 feet) were considered in the development of the method. The analyses were used to determine average acceleration histories on pre-determined slide masses. These histories were then used in Newmark analyses to estimate the resulting displacement of the masses. A series of charts were developed from the analysis results for use in predicting seismically-induced displacements of embankments. Although this analysis considers the dynamic response of the embankment, the evaluation of sliding is still decoupled from the estimate of dynamic response. All displacements are also assumed to occur on a single sliding plane.

This method is considered adequate for the preliminary analysis of compacted clay or dense sand embankments having a moderate height and non-yielding foundation. The analysis does not apply to structures having materials susceptible to significant increases in pore pressures or loss in strength due to cyclic loading.

The Makdisi and Seed approach includes the following steps:

(1) Determine k_y using the limit equilibrium method (e.g. UTEXAS4 with the Spencer method of analysis). Failure surfaces that disrupt the crest and involve progressively larger portions of the embankment should be considered (e.g., critical surfaces that pass through the upper fourth, upper half, and the full embankment may be analyzed). The *y/h* ratio for each failure surface is determined, where *y* is the maximum depth of the sliding mass and *h* is the height of the embankment.

The yield acceleration should be calculated using soil strengths that are appropriate for rapid cyclic loading. The consolidated undrained strength is generally the most realistic for saturated materials of relatively low permeability. It may be appropriate to limit the undrained strength of dense, dilatant material to no more than the drained strength. Drained strengths are typically used for unsaturated materials and for very pervious materials with unobstructed drainage (e.g. clean gravels at the ground surface). For compacted clayey materials, Makdisi and Seed recommend using 80% of the yield strength measured in typical monotonic shear tests.

[For example, assume k_y = 0.15g, y/h = 0.5, M = 7.5, and PGA = 0.3g.]

(2) The graph shown in Fig. 9 is used to determine the ratio k_{max} / u_{max}, where k_{max} is the maximum average acceleration for the potential sliding mass extending to depth *y* and u_{max} is the maximum crest acceleration.

[For y/h = 0.5, $k_{max} / u_{max} \approx 0.75$ for upper bound]

(3) The maximum crest acceleration u_{max} may be estimated using a number of techniques. Makdisi and Seed developed a hand-calculation procedure based on a simplifying assumption regarding the structural behavior. A finite element analysis may also be performed, although this defeats the simple nature of this approach. Alternatively, trends of u_{max} versus base acceleration derived from actual recordings of transverse crest acceleration may be used, such as shown in Fig. 10.

[For PGA = 0.3g, $u_{max} \approx 0.6$g for upper bound]

(4) Calculate the maximum average acceleration k_{max} for the potential sliding mass from steps 3 and 4.

[For $k_{max} / u_{max} \approx 0.75$ and $u_{max} \approx 0.6$g, $k_{max} = 0.45$g]

(5) Calculate k_y / k_{max} from steps 1 and 4.

[For $k_y = 0.15$g and $k_{max} = 0.45$g, $k_y / k_{max} = 0.33$]

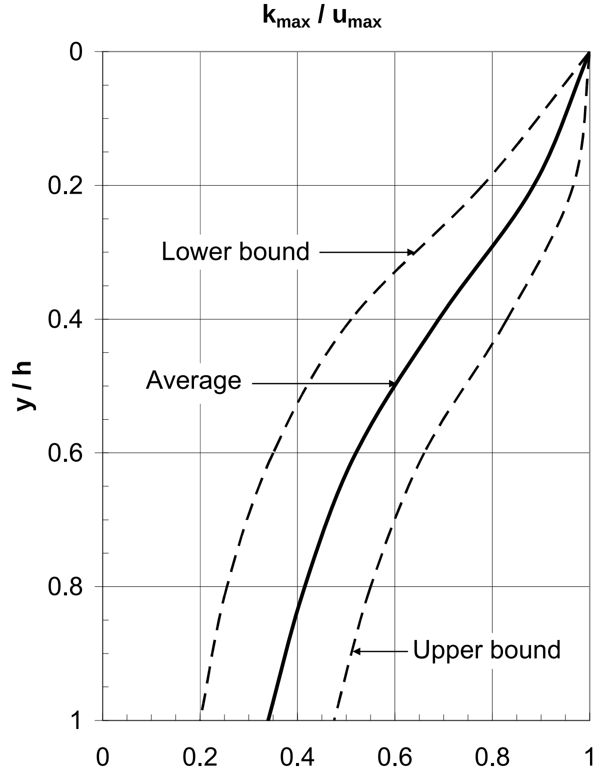


Fig. 9. Variation of maximum acceleration ratio with depth of sliding mass (based on Makdisi and Seed 1978).

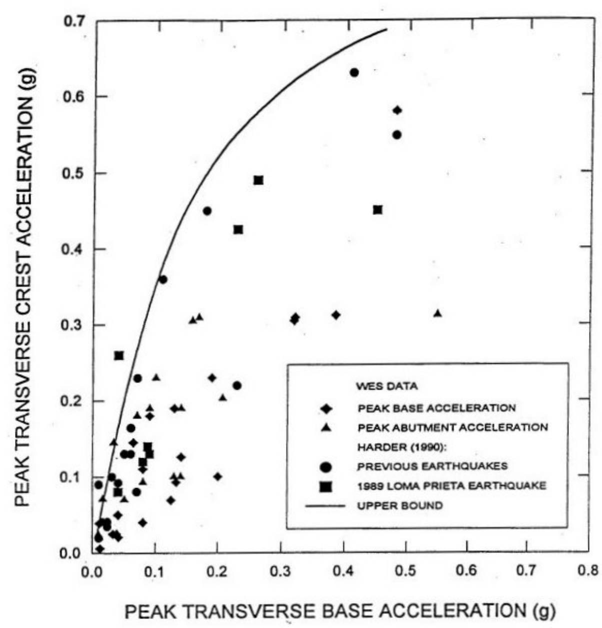


Fig. 10. Comparison of peak base and crest transverse accelerations measured at earth dams (from US Army Corps of Engineers, 2000).

(6) For the expected earthquake magnitude and the known k_y / k_{max} ratio, use the graph in Fig. 11 to estimate the range in potential permanent displacement. These ranges are given for three earthquake magnitudes, as magnitude is generally related to duration of ground shaking and, consequently, more acceleration cycles exceeding the yield acceleration.

[For M = 7.5 and k_y / k_{max} = 0.33, u = 60 cm]

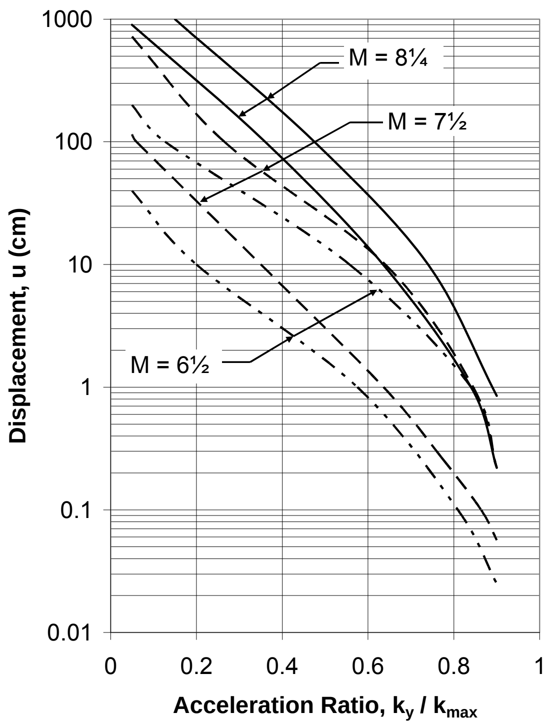


Fig. 11. Variation in permanent displacement with yield acceleration (based on Makdisi and Seed 1978)

There are a number of restrictions or limitations to the Makdisi and Seed procedure that must be regarded when the approach is used for an initial estimate of displacement. The procedure only applies to dams built of materials experiencing little or no strength loss during earthquake shaking, such as well-compacted sands or clays. Due to the assumption of no strength loss during shaking, and the application of equivalent linear principles in the underlying analyses, the procedure may be questionable for severe ground shaking. In addition, the analyses used to develop the curves were based on a very limited set of earthquake records and dam geometries that may not suitably represent a specific site. The approach is also very sensitive to the estimate of crest acceleration (Bureau 1997).

Category 3 – Post Earthquake Limit Equilibrium Analysis

Limit equilibrium analysis can be used for the preliminary evaluation of deformations including the effects of liquefaction. This method assesses the stability of the dam for gravity loads after any strength degradation caused by the earthquake. The analysis is most useful for identifying cases where the anticipated deformations are large and controlled by general instability. In these cases, the displacements that occur during the earthquake tend to be implicitly addressed in the evaluation. In some cases this approach may be used to estimate the final deformed shape of the dam.

The following steps are used to perform the analysis:

(1) The potential for liquefaction and development of excess pore pressure in the embankment and foundation is evaluated using a standard methodology. The simplified 1-D Seed-Idriss procedure can sometimes be used for preliminary estimates, although these results should be considered very approximate for dam evaluations. (Youd et al. 2001). A site response analysis may also be performed using a 1-D or 2-D computer program to model the propagation of earthquake motions through the foundation and embankment, such as SHAKE (Schnabel et al. 1972) or QUAD4M (Hudson et al., 1994). 2-D analyses are generally preferred. The predicted cyclic shear stresses are then compared to estimates of the available cyclic shear resistance obtained from empirical formulations or laboratory tests. Evaluating the response of moderately plastic soils is addressed by considering the work of Bray and Sancio (2006) and Boulanger and Idriss (2004). Refined studies will generally require laboratory testing of these soils to better define their anticipated behavior.

(2) Appropriate post-earthquake shear strengths should be selected for zones identified to liquefy or experience strength loss. For liquefaction, empirical estimates of the residual strength S_r as back-calculated from documented flow failures are typically used. These empirical estimates are formulated in terms of the residual strength S_r (e.g., Seed and Harder, 1990; Idriss and Boulanger, 2007) or a normalized strength S_r/σ'_{vo} (e.g., Olson and Stark, 2002; Idriss and Boulanger, 2007). The Corps does not currently prefer one approach over the other, and both may be considered in a sensitivity study of critical facilities. Some soils may also be appropriate for laboratory testing of post-earthquake strength, such as fine-grained soils with low permeability.

(3) Post-earthquake shear strengths for non-liquefied zones should still consider the generation of residual excess pore pressures. Relationships between the factor of safety for liquefaction and residual excess pore pressure may be used (Marcuson et al., 1990, see Figure 16). The strength of cohesive soils that may suffer significant shear strains should be reduced. If large strains are anticipated, the strength should be reduced to either the remolded or residual strength value.

(4) The limit equilibrium analysis is typically performed using Spencer's method and an appropriate computer program (e.g., UTEXAS4 by Wright, 1999). Both circular and wedge-type surfaces should be considered. Large deformations are considered likely if the computed factor of safety is less than one. Moderate deformations are expected as the safety factor rises above 1. Safety factors in excess of 1.2 to 1.5 may be required to achieve tolerable displacements in many cases. The critical safety factor depends on a number of factors including the severity of the earthquake loading. For cases where the Factor of Safety exceeds 1, it is helpful to compute the corresponding value of the yield acceleration k_y. Newmark analyses show that dynamic displacements tend to be directly related to yield acceleration. Although the computed value of k_y assumes the full anticipated strength loss from the earthquake, it can still be a useful parameter in gauging the anticipated displacements.

(5) The critical sliding surface identified in the limit equilibrium analysis is often a reasonable approximation of the anticipated sliding surface. The final stable geometry may be approximated through a series of limit equilibrium analyses. The geometry in each analysis step is adjusted to reflect a modest movement along the critical failure surface defined in the previous step. This progressive analysis with evolving geometry is continued until a safety factor in excess of 1.0 is achieved. Seed et al. (2003) recommends this analysis should be continued until the factor of safety exceeds 1.05 to 1.2 to account for momentum effects of the sliding mass. The appropriate safety factor depends on the anticipated velocity of the sliding mass during failure and whether inertial effects are indirectly considered in the estimate of S_r.

The effect of momentum on each of the flow failure case histories is not always clear. Olson and Stark (2002) back-analyzed 33 case histories and were able to consider inertial effects in 10 of these cases. They also concluded that the strength derived from 22 of the remaining case histories were not significantly affected by momentum effects due to the relatively small heights involved (i.e., slope heights less than 10 m).

<u>Category 3 – Success Dam Example:</u> A conservative procedure was used for a preliminary evaluation of Success Dam with a proposed seismic retrofit (Perlea et al. 2008). This remediation option, currently under final evaluation, includes the following steps:

- Excavation of a portion of the downstream shell to expose as much liquefiable foundation material as possible while leaving the dam in a stable condition.

- Complete removal of liquefiable alluvium beneath the footprint of a new core and shell zone downstream of the existing dam.

- Construction of a new core, transition zones, and shells downstream of the existing dam.

- Degrade crest of existing embankment and use materials to construct upper portions of new zones.

The upstream portion of the existing dam is left in place and is considered to act as a sacrificial buttress. The initial assessment of this plan included a conservative stability evaluation based on the post-earthquake limit equilibrium analysis. Because of the preliminary nature of this evaluation, two simplifying and conservative assumptions were made: 1) any material capable of liquefying was assigned an appropriate residual strength, and 2) materials within the upstream sacrificial zone that slide away from the primary dam are no longer considered as having a buttressing function. The results of this analysis are shown in Fig. 12 and were used to define the initial limits of the sacrificial zone.

Although most of the existing embankment is left in place, the upstream toe is considered sacrificial under the maximum credible earthquake with the condition that the buttress is not jeopardized by a progressive failure. This preliminary evaluation built confidence that this remediation variant was a viable proposal since the progressive failure is expected to stop before significantly impacting the buttress.

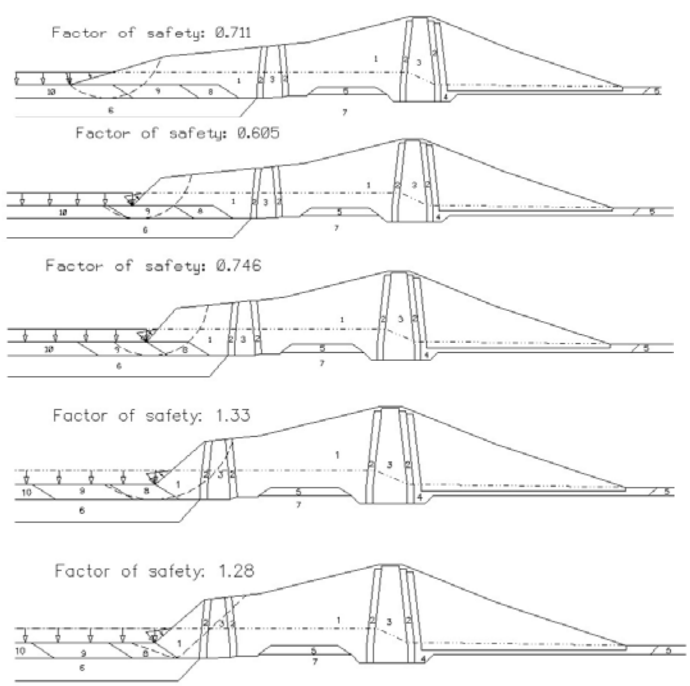


Fig. 12. Simplified evaluation of progressive failure at the remediated Success Dam. Broken lines indicate potential failure surfaces with factors of safety as noted. (Perlea et al. 2008)

Category 4 – Hybrid approaches

Attempts have been made to combine aspects of the Category 2 and 3 analyses while still maintaining a simplified procedure. The most common may be to perform a Newmark analysis while using the anticipated post-earthquake strengths for estimating the yield acceleration. While these analyses may improve displacement estimates in some situations, the complexity involved in appropriately performing such an analysis often precludes its use for simplified evaluations. For example, performing a Newmark analysis with residual strengths mobilized at the beginning of the earthquake will not necessarily provide a conservative estimate of displacements. Dynamic response issues, as well as plastic soil response at stresses below yielding, can significantly impact the accumulation of displacements when significant liquefaction occurs. Seismic response histories may also change significantly above any liquefied zone. Analyses of this category are not recommended when substantial liquefiable materials are present.

ADVANCED METHODS

Advanced analysis methods cover a wide range in sophistication and complexity. The intent of these methods is to address some of the key issues that are not adequately considered in the simplified approaches. A general list of typical features is provided below. The most sophisticated analyses will address all of these issues to some level of success, while the simpler analyses will only incorporate a few of these features.

- The dynamic response of the structure is modeled, typically using a 2-D finite element or finite difference numerical model.
- Non-linear soil behavior is considered, either through equivalent linear approximations or through hysteretic stress-strain models.
- Pore pressures are generated from cyclic shear loading.
- Material properties are affected by estimated changes in effective stress and/or strain-softening behavior.
- Plasticity models are used to permit estimates of deformations and strains through yielding.
- The dilative and/or contractive nature of the soil response is directly modeled.
- The coupling between dynamic response and material softening and yielding is directly considered.
- Pore water flow and continuous re-distribution of pore pressure during and after shaking are considered.

A key aspect of any advanced analysis, especially those involving greater complexity and sophistication, is the development and review of adequate documentation. Particular attention should be given to formally documenting the behavior of the selected constitutive models. Typical stress strain behavior of the models under the anticipated loading conditions should be clearly demonstrated. The model response in terms of secant stiffness and hysteretic damping versus strain should be demonstrated and compared to experimental results or published trends. The ability of the model to correctly model liquefaction under the shear and effective stress regime of the dam should be demonstrated. Post-liquefaction behavior, including loss of strength and stiffness, should be demonstrated. Results from a back-analysis of a case history using the model should also be available and reviewed as part of the analysis of any critical embankment.

A number of programs have been used by the Corps of Engineers or their contractors for successful evaluation of embankment dams to seismic loading. These programs range from those using simple equivalent linear techniques (QUAD4M), to sophisticated nonlinear elastic analyses (TARA-3, TARA-3FL), to complex plasticity-based programs (DYNAFLOW, FLAC). The finite difference program FLAC has been applied with a range of user-defined constitutive models. The following presents an overview of a number of recent analyses performed on Success Dam and Tuttle Creek Dam using this wide range in analytical tools. Additional details on each program, the embankments, and the analysis results can be found in the referenced documents.

QUAD4M – Equivalent Linear Method

The equivalent linear method can be used to perform a 2-dimensional site-response analysis of the embankment. Although the method performs a linear elastic analysis of the structure, the stiffness and damping properties are iteratively adjusted to be compatible with the estimated cyclic shear strains. The method is particularly useful for estimating zones of liquefaction within the embankment and foundation as well as predictions of peak cyclic shear strain. It may also be used to perform the same type of response and displacement estimate as performed by Makdisi and Seed (1978) in the development of their simplified method.

The QUAD4M program is currently used by the Corps to perform 2-dimensional equivalent linear analyses (Hudson et al. 1994). The Q4MESH software was developed by the Corps of Engineers Waterways Experiment Station WES (currently the Engineer Research and Development Center, ERDC) in cooperation with Sacramento District and is used to facilitate post-processing of the results. The Q4MESH program has the ability to estimate cyclic stress ratios (CSR) based on the peak shear stresses that occur in each element. Utilizing the calculated CSR values, the factor of safety against liquefaction is calculated using criteria set forth by Youd et al. (2001).

The primary advantage of the equivalent linear method is its simplicity of operation and input, and the wide experience of its use in the profession. While the technique provides a rational estimate of the dynamic response of the structure, it does not directly consider such effects as pore pressure generation, plastic yielding, or pore water flow. Although it does not directly estimate the seismic deformations, it is routinely used by USACE as an aid in evaluating results furnished by more sophisticated models.

For the remediated Success Dam, as shown above in Fig. 12, the liquefaction susceptibility for MCE loading was estimated using the computer program QUAD4M. The CSR estimated from QUAD4M is shown in Fig. 13. The cyclic resistance ratio (CRR) was evaluated using the $N_{1,60}$ blowcount data from the Standard Penetration Test investigations as described by Idriss and Boulanger (2004). CRR was calculated in the liquefiable soil only and is shown in Fig. 14.

The factor of safety against liquefaction, presented in Fig. 15, was defined by the ratio CRR / CSR after correction for magnitude (MSF) and overburden stress (K_σ). To simplify the QUAD4M analysis, the static shear stress correction factor (K_α) was conservatively assumed equal to unity.

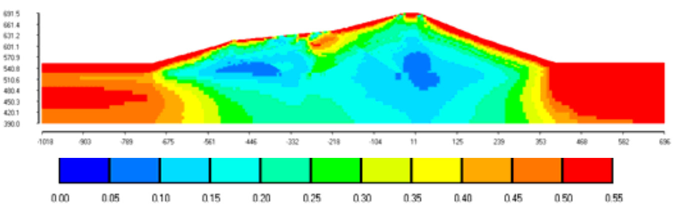


Fig. 13. CSR calculated from 65% of peak QUAD4M element shear stress. (Perlea et al. 2008)

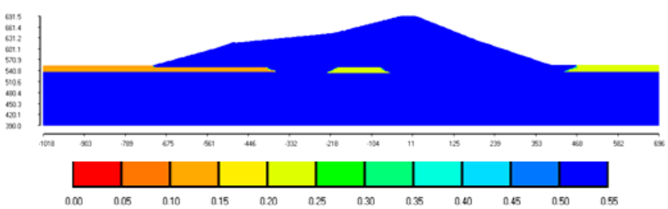


Fig. 14. CRR estimated from $N_{1,60}$. (Perlea et al. 2008)

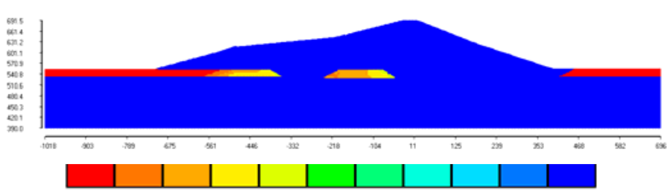


Fig. 15. Factor of safety against liquefaction. (Perlea et al. 2008)

Excess pore water pressures were also approximated from the liquefaction factor of safety. The selected relationship between excess pore pressure ratio and factor of safety was taken as the average gravel curve from data presented by Marcuson et al. (1990) and shown in Fig. 16. Excess pore pressure ratio r_u is defined as the ratio of the excess pore pressure generated by the cyclic loading to the initial vertical effective stress. The estimated contours of r_u are shown on Fig. 17.

The evaluation shows that a portion of the existing upstream embankment, upstream recent alluvium, and downstream toe alluvium have the potential of reaching a liquefied state during the MCE. The recent alluvium between the new and existing cores is predicted to experience a more modest increase in pore pressures, with r_u values between 10% and 50%.

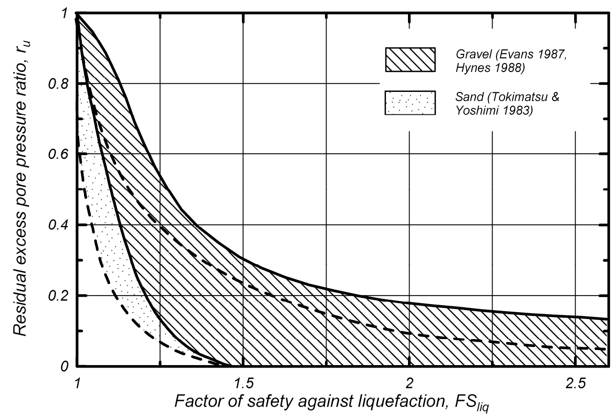


Fig. 16. Relationship between the excess pore pressure ratio and the factor of safety against liquefaction triggering (Marcuson et al. 1990)

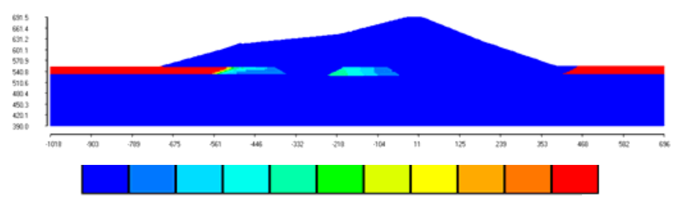


Fig. 17. Percent r_u estimated from QUAD4M evaluation (Perlea et al. 2008)

<u>TARA-3 – Nonlinear elastic effective stress analysis</u>

TARA-3 uses an effective stress approach in a 2-dimensional finite element analysis. The constitutive model is based on direct modeling of the nonlinear hysteretic stress-strain response of the soil to static and cyclic loading. Simulation of pore pressure increases during shaking is related to estimates of the development of plastic volumetric strains. Changes in effective stress due to increases in pore pressure directly affect the element stiffness. The model also takes into consideration the effects of the initial static shear stress on the pore pressure build-up.

Professor W. Liam Finn, the primary developer of TARA-3, performed a seismic analysis of Tuttle Creek Dam using this program (Finn 2004). The analysis predicted a crest settlement of about 50 feet, with a horizontal downstream movement of about 25 feet, due to loading from the MCE. The predicted displacements of the upstream and downstream toes were also large, exceeding 45 feet, as shown on Fig. 18. The predicted generation of pore pressures in selected elements is shown on Fig. 19.

Although most of the loose sand in the foundation was predicted to liquefy, the displacements occurred primarily by shearing within the upper cohesive blanket of the foundation soil. This blanket is 15 to 20 feet in thickness (3 layers of elements upstream and 4 layers of elements downstream on the finite element mesh shown in Fig. 18) and consists of silts and low plasticity clays. For modeling the strength loss and softening of this cohesive blanket, the constitutive model for sand was calibrated to improve the match between model behavior and cyclic triaxial test results.

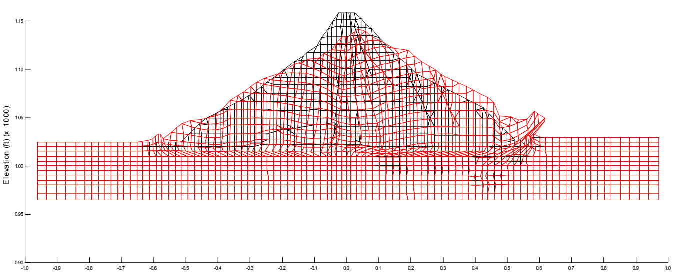


Fig. 18. Post-liquefaction deformed shape of Tuttle Creek Dam; scale 1H = 3V (Finn 2004).

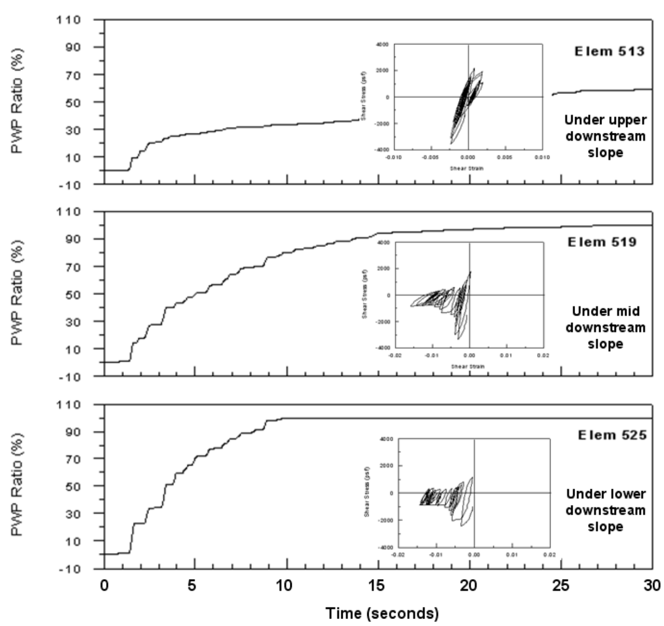


Fig. 19. Excess pore pressure increase in liquefiable sand during shaking (20 s) and thereafter. (Finn, 2004)

TARA-3FL – Nonlinear elastic analysis for post-earthquake

TARA-3FL is an extension of TARA-3 for performing static post-earthquake deformation analysis. This program was used by WES (currently ERDC) to evaluate the post-liquefaction behavior of Tuttle Creek Dam (WES, 2000). The entire liquefiable sand layer and/or sensitive cohesive blanket were assumed to degrade in strength and stiffness. This study concluded that significant deformations can occur due to strength reductions in either the liquefiable sand or sensitive cohesive soil. The computations showed that deformations were very sensitive to small changes in pore pressure after the excess pore pressure ratio in the foundation reached about 50% - 60%. Fig. 20 presents the predicted deformations when r_u reaches about 80% in the liquefiable sand. Similar results were obtained when pore pressures were increased in the cohesive soil. The TARA-3FL results generally confirm the conclusions from the TARA-3 analysis, although the potential displacements are shown to be more severe.

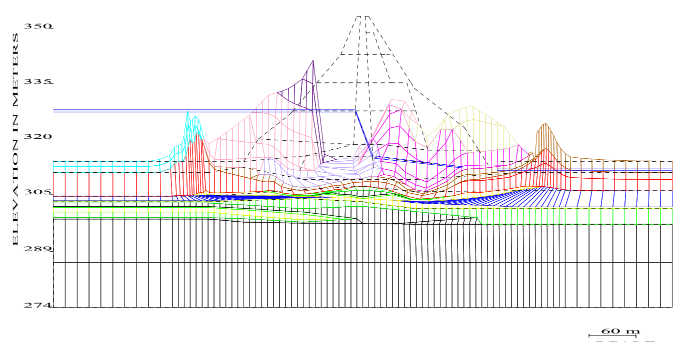


Fig. 20. Predicted deformation by TARA-3FL of Tuttle Creek Dam for r_u = 80% in the liquefiable sand. Magnification factor for deformed shape = 1.0. (WES, 2000).

DYNAFLOW – Plasticity-based effective stress analysis

DYNAFLOW is a sophisticated two-dimensional finite-element program based on plasticity principles and developed by Prof. Prevost (Prevost, 1981; Prevost, 1998). It is capable of performing nonlinear dynamic analyses of embankment dams under seismic loading. The computations are conducted in terms of effective stress, using fully coupled solid-fluid equations for the treatment of saturated porous media. Among several available constitutive models, a multi-yield surface plasticity model has been used for modeling embankment and foundation materials, including liquefiable sands. The multi-yield constitutive soil model is a kinematic hardening model based on plasticity theory and is applicable to both cohesive and cohesionless soils. It simulates the material hysteretic behavior, the shear stress-induced anisotropic effects, and strain hardening due to dilation. DYNAFLOW is proprietary to Princeton University and is available by lease.

DYNAFLOW was used in early stages of the seismic analysis of Tuttle Creek Dam (Popescu 1998). Fig. 21 shows selected results from the analyses based on conservative estimates of blowcount. At the end of the MCE shaking, having a duration of about 20 seconds, the horizontal displacements were on the order of about 35 feet at the upstream toe and 25 feet at the downstream toe. Crest settlements were only 1.5 feet. Fig. 22 shows displacement histories at the toes of the dam both during and after the period of strong shaking. While the displacements predicted using average estimates of blowcount (solid line) are relatively small and stabilize after the earthquake, the predictions based on conservative estimates of blowcount (dashed line) show unstable conditions. Significant movements continue to occur at the end of the analysis (at about 21 seconds). The analysis was not continued beyond this point since it would have required re-meshing of the severely distorted grid.

<u>FLAC – Plasticity-based finite difference program</u>

FLAC, or Fast Lagrangian Analysis of Continua, is based on the explicit finite difference method for modeling nonlinear static and dynamic problems. The program is capable of performing effective stress analyses with full coupling to the fluid flow solution. The program uses a Lagrangian procedure to update the mesh geometry in cases of large deformation. FLAC contains a number of general purpose constitutive models, such as the elastic-plastic Mohr-Coulomb model, but it also provides for the use of user-defined constitutive models. Three such models are discussed below: the Wang model, the URS model, and UBCSAND.

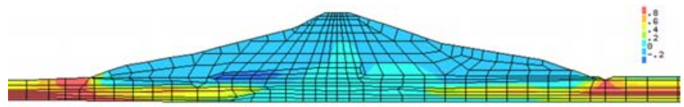
The FLAC program and documentation is available from the Itasca Consulting Group (Itasca, 2008).

<u>FLAC with Bounding Surface Hypoplasticity model</u>

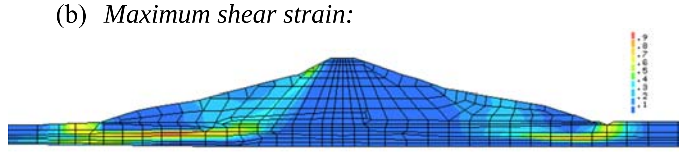
AMEC Geomatrix, Inc. (formerly Geomatrix Consultants, Inc.) uses FLAC with a user-defined constitutive model for sand that is based on bounding surface hypoplasticity (Wang et al., 1990). The model was implemented in a 2-D version of FLAC (Wang and Makdisi, 1999). The stress-strain relationship is fully nonlinear under both loading and unloading conditions. The model is capable of simulating volumetric changes induced by increments of shear stress and the corresponding variation in pore pressures.

The Wang model that was applied to the evaluation of Success Dam used both the MCE (PGA = 0.22g) and the operating basis earthquake, OBE (PGA = 0.10g) with a duration of about 80 seconds. Complete liquefaction of the recent alluvium in the foundation was predicted for both earthquake scenarios. Example element predictions are shown on Fig. 23. The dilative behavior captured by the constitutive model is seen to contribute to fluctuations of mean effective stress after the triggering of liquefied behavior. The minimum allowable effective mean stress was set at 0.3 ksf in the model, which defined the triggering of liquefaction.

(a) *Excess pore pressure ratio, r_u:*



(b) *Maximum shear strain:*



(c) *Contours of horizontal displacements.*

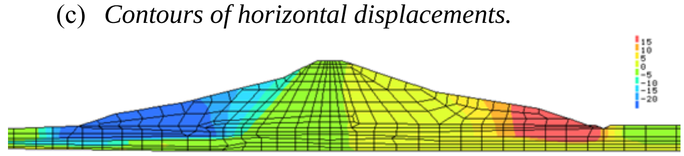


Fig. 21. Selected DYNAFLOW analysis results at the end of strong shaking for Tuttle Creek Dam (Popescu, 1998)

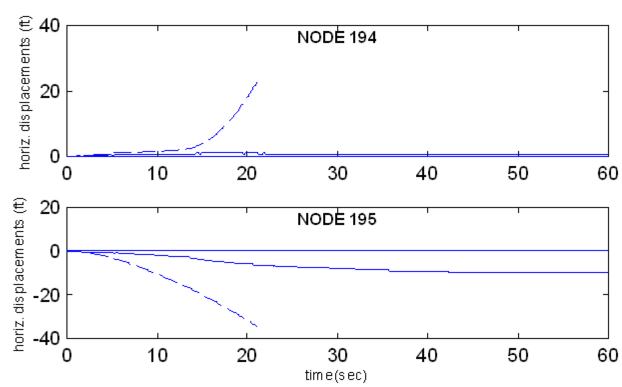


Fig. 22. Continuing deformation of the toes after the end of shaking: node 194 is at the upstream toe; node 195 is at the downstream toe; solid lines: average parameters; broken lines: 33-percentile SPT $N_{1,60}$. (Popescu, 1998).

At the time of this study (2003), the corrected SPT blowcount $N_{1,60}$ was of the alluvium was considered to be 10 in the zone upstream of the core and 15 downstream of the core. Geomatrix emphasized the importance of modeling the dilation behavior of the recent alluvium as shown by triaxial laboratory tests results (Fig. 24). The analysis predicted large deformations of the upstream shell as shown in Fig. 25.

Based on the finding that the entire layer of recent alluvium beneath the dam is expected to liquefy, FLAC was also used to perform a pseudo-static deformation analysis of the post-earthquake condition of the embankment. The liquefied material was assigned a strength equal to the estimated S_r, and the corresponding shear modulus was estimated using the limiting strain concept from Seed et al. (1985). The post-liquefaction shear modulus was taken as the ratio of the residual strength to the limiting strain, as illustrated in Fig. 26.

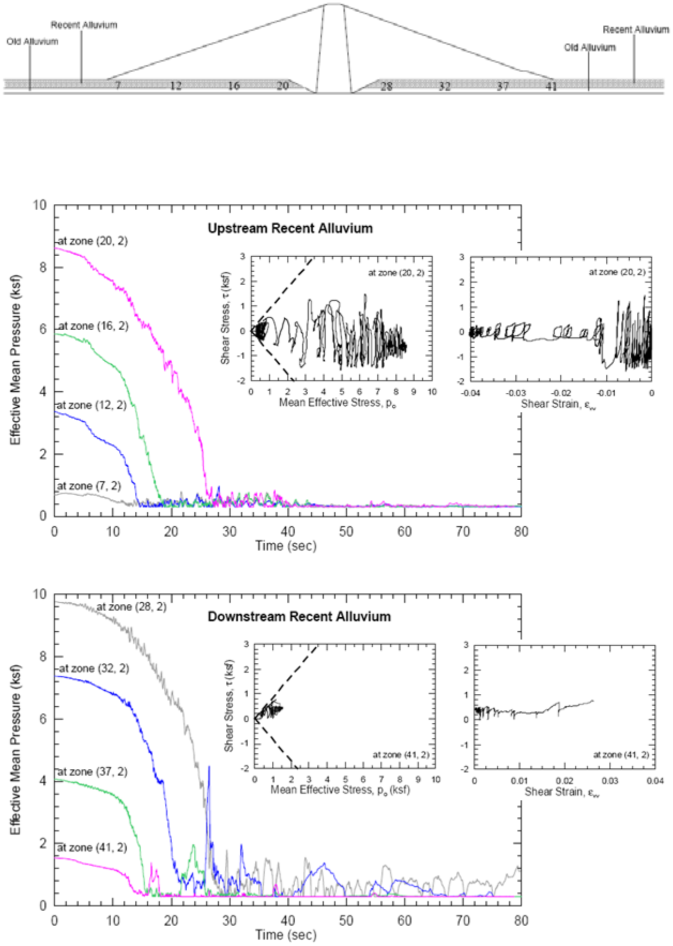


Fig. 23. Computed stress histories and stress paths in selected location of the recent alluvium of Success Dam as computed by the Wang model (Geomatrix, 2003).

The post-earthquake analysis resulted in the deformed shape shown in Fig. 27. The deformations shown in this figure are not the final maximum values since the model was still deforming when the analysis was stopped due to excessive distortion.

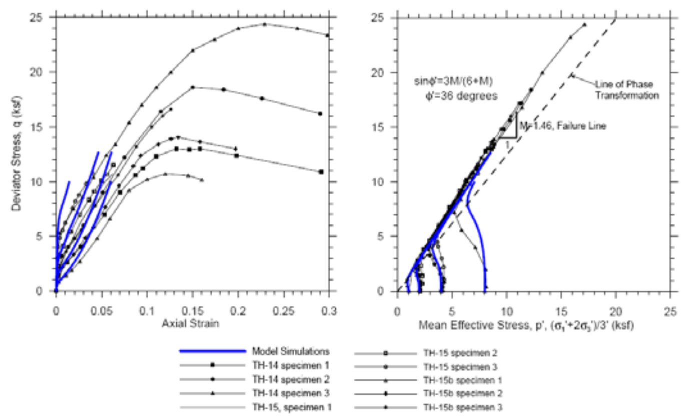
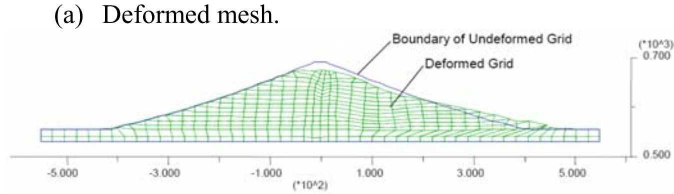


Fig. 24. Comparison of Wang model simulations and triaxial test results (Geomatrix 2003).

(a) Deformed mesh.



(b) Contours of horizontal displacement in feet.

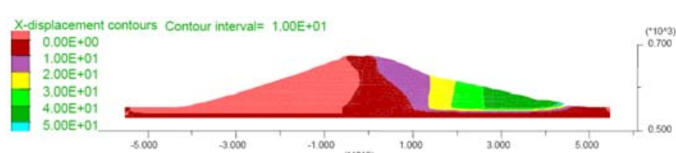


Fig. 25. Estimated results for Success Dam at the end of MCE shaking from Wang model (Geomatrix, 2003).

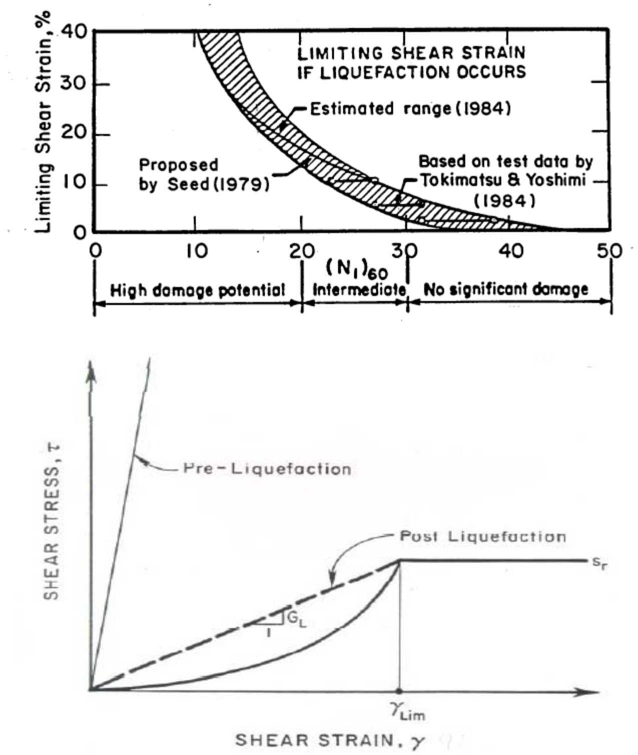
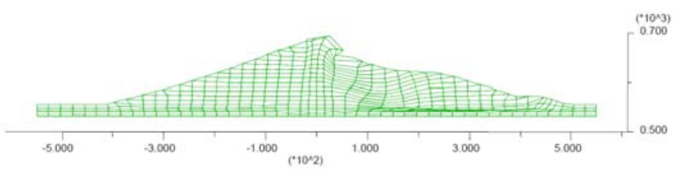


Fig. 26. Modulus relations used in Geomatrix pseudo-static deformation analysis (Geomatrix 2003).

(a) Deformed mesh.



(b) Horizontal displacement contours.

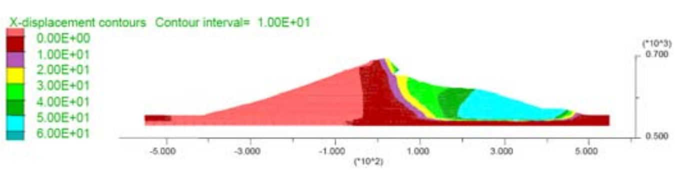


Fig. 27. Estimated results for Success Dam at end of post-earthquake analysis. Note: structure not at equilibrium at end of analysis. (Geomatrix, 2003).

FLAC with URS cycle-weighting model

Early evaluations of Success Dam and retrofit variants were performed for the Corps of Engineers by URS Corporation using a relatively simple effective stress model. This approach couples the standard elastic-plastic Mohr-Coulomb model to an empirical pore-pressure generation scheme (Dawson et al. 2001). The pore pressure is incrementally generated during shaking through a cumulative damage approach. Each time a half-cycle of shear stress is detected, the model computes the cyclic stress ratio for that cycle and determines the corresponding increment of pore pressure change using a specified cycle stress curve (i.e., a curve of number of uniform load cycles to liquefaction versus CSR). The available shear strength decreases with increasing pore pressure until a minimum value equal to the residual strength is achieved. This is illustrated in Fig. 28.

Although relatively simplistic when compared to other constitutive models, the predicted deformations and zones of liquefaction often compared well with those estimated by a more sophisticated model. Fig. 29 compares results obtained with the URS model to those obtained with the UBCSAND model. The UBCSAND analyses are described in the next section.

The extent and pattern of displacements predicted by the two models are similar, as well as the general conclusion that major deformations should be expected. The simpler URS model predicted larger displacements at the crest and the upstream slope, but smaller at the downstream toe. One of the major differences between the two models is in the prediction of excess pore pressures. An example of the predicted pore pressure histories is illustrated in Fig. 30.

Although Fig. 30 shows good agreement in excess pore pressures for this case, there is a significant difference in the computed trends. The URS model predicts pore pressures that increase monotonically until the maximum value is reached. In contrast, UBCSAND analysis considers both the dilative behavior of soil and its effect on pore pressures, as well as the potential for pore pressure dissipation during the earthquake.

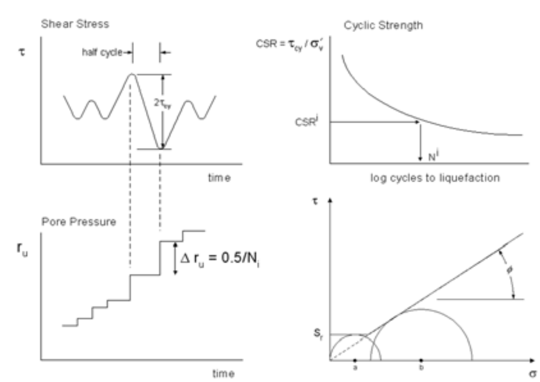


Fig. 28. Schematic showing basic principles of URS constitutive model (after E, Dawson, private comm.).

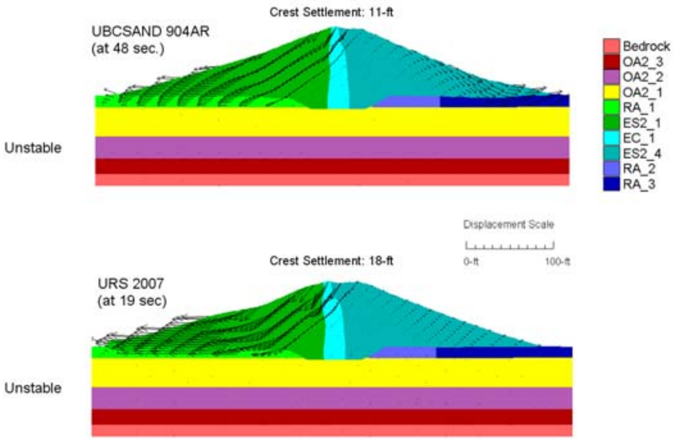


Fig. 29. Deformed shape and displacement vectors after MCE action on a cross section of Success Dam; reservoir with low, conservation pool on the left side (after E, Dawson, private communication).

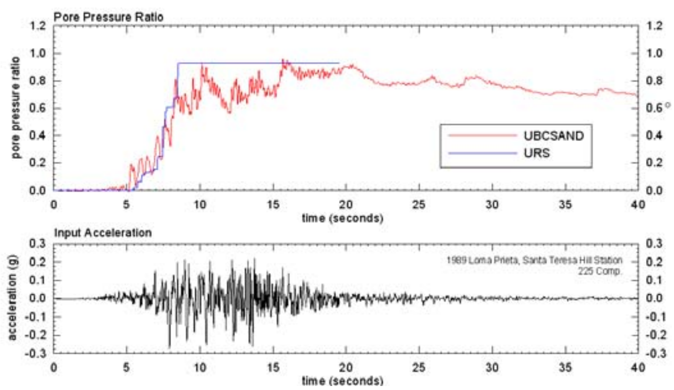


Fig. 30. Accelerogram and corresponding pore pressure build-up time history in the middle of the liquefiable material marked RA_1 in Fig. 29 (after E. Dawson, private communication).

FLAC with UBCSAND model

UBCSAND is a modified Mohr-Coulomb model that directly assesses plastic shear and volumetric strains during every loading step. Each increment of plastic volumetric strain is directly related to the current stress ratio, the increment of plastic shear strain, and the cyclic stress history. For saturated soil elements, the tendency for contraction of the soil skeleton increases the pore pressures while the tendency for dilation decreases the pore pressure. The model incorporates a hyperbolic relationship between stress ratio and plastic shear strain. Unloading is linear elastic, so hysteretic stress-strain loops are produced during cyclic loading. The model reproduces the 'banana shaped' stress-strain loops after liquefaction as often observed in laboratory cyclic tests with dilation hardening and contraction softening. One advantage of a UBCSAND-type model over simpler approaches, such as the URS cycle-weighting method, is the prediction of stress-strain and stress path response which begins to resemble the intricate behavior observed in laboratory test results.

UBCSAND does not typically limit the post-liquefaction strength of an element to the estimated residual strength. It is possible for liquefied zones to mobilize strengths greater than the residual strength during shaking. Instead, a UBCSAND seismic analysis typically includes a post-earthquake stability evaluation using the Mohr Coulomb model and residual strengths in zones that liquefied during shaking. This analysis is performed on the model at the end of shaking. After converting the strengths to residual strength, the analysis is continued in dynamic mode until the model reaches equilibrium.

FLAC and UBCSAND was used by the Corps of Engineers to evaluate the seismic deformation of Tuttle Creek Dam after the final site characterization was completed (Stark Consultants, 2007). The FLAC Mohr-Coulomb was used in the non-liquefiable embankment. Two constitutive models were used to represent the foundation soils, both of them originally developed at the University of British Columbia, Canada: UBCSAND (Byrne et al., 2003) and UBCTOT (Beaty and Byrne, 2008). UBCSAND was used for the liquefiable sands in the foundation, while a modified version of UBCTOT was applied to the cohesive soil layer.

UBCTOT is a total stress model that simulates the triggering of liquefaction and mobilization of residual strengths. The version used for Tuttle Creek Dam was modified to include a hyperbolic stress-strain response for elements that have not liquefied. The tangent stiffness determined from the hyperbolic relationship is further modified to account for the predicted generation of cyclically-induced pore pressures. The onset of liquefaction is estimated using a cumulative damage technique where the effect of each irregular cycle of shear stress is combined and compared to the laboratory test results using a weighting curve. The post-liquefaction response is modeled using a simple bilinear representation of the stress-strain relationship during loading. The model was calibrated to laboratory cyclic triaxial tests on undisturbed samples (Castro et al. 2003) supplemented by large strain undrained strength from field vane shear tests.

Five columns of foundation soil with different properties were defined based on the results of field investigations: under the crest, mid-slope, toe, and free field as shown on Fig. 31. The upper layer of the foundation soil was modeled with UBCTOT while the lower layers were modeled with UBCSAND.

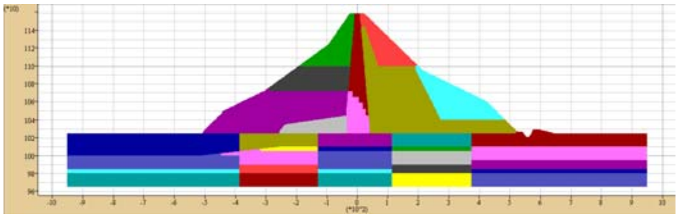


Fig. 31. Materials zones for FLAC-UBCSAND analysis of Tuttle Creek Dam (deformed scale).

Figure 32 presents the predicted shear strain contours after the design earthquake. The contours suggest both the upper cohesive layer and lower foundation sands play a role in the deformations, with liquefaction of the sands being most important near and below the downstream toe. Maximum displacements of the downstream shell were predicted to be less than about 4 feet. It is evident that the refinements made in the site investigation and subsequent modeling led to a significant reduction in the predictions. However, the analysis also showed that the estimated displacement of the downstream shell could increase significantly with rather small changes in loading or input parameters. These conclusions have led to the current remediation of the dam.

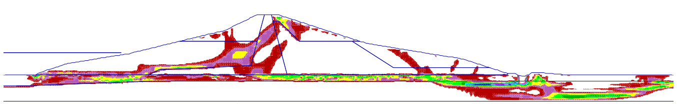


Fig. 32. Shear strain contours predicted for Tuttle Creek Dam by FLAC with UBCSAND and UBCTOT models (Stark Consultants, 2007).

<u>General Conclusions</u>

The range of analysis methods and corresponding results used for Tuttle Creek Dam and Success Dam are not directly comparable even for those analyses performed on the same structure. Input parameters often changed between studies due to investigations performed during the sequence of analyses. Final evaluations often incorporate more detailed analyses than used in preliminary assessment. However, this set of analyses does lead to some conclusions of general interest for performing advanced deformation evaluations:

- When advanced deformation estimates are required, the analyses should be based on a dynamic response analysis that couple inertial effects and material yielding. Consideration should be given to pore pressure migration during and after shaking and to the effects of material dilation during shear strain.

- The dynamic behavior of plastic soil requires special consideration, particularly for sensitive clays and low to moderate plasticity silts and clays. Laboratory testing of undisturbed samples is recommended in many cases.

- Advanced analyses require a well-planned parametric study to evaluate the fragility of the analysis and structure.

- The appropriateness of selected constitutive models should be demonstrated by comparing laboratory test simulations with laboratory data and/or empirical relationships. These comparisons should include (as appropriate) modulus reduction with strain, damping, liquefaction triggering under an appropriate range of effective stress and initial shear stress conditions, post-liquefaction stress-strain behavior, and strain-softening for sensitive materials.

- Displacement estimates can significantly change as additional site investigations are performed and refinements are made to the deformation analysis. While some advanced deformation analyses can be useful during the initial phases of an evaluation to help identify key aspects of the response, it is often efficient to delay a full suite of detailed analyses until the site characterization is finalized and documented.

PREFERED APPROACH (2010)

The methodology recommended for evaluation of Corps dams depends primarily on the stage of analysis and the corresponding level of effort. Guidance on this phased approach is currently being revised and will be included in the Corps engineering manual on seismic analysis of embankment dams which is currently under development. The following discussion will provide some additional details on the more advanced methodology.

<u>Earthquake Loading</u>

The seismicity study is performed in the early stages of the deformation analysis, although additional development of the loading may be required for subsequent stages. The design earthquake(s) should be defined through its magnitude and site-specific response spectra. This may include definition of a deterministic MCE as well as a probabilistically-derived OBE (as required by ER 1110-2-1806 and/or the projected replacement by Engineering Circular currently being developed by AMEC Geomatrix).

Ground motion histories are required for the numerical deformation analysis. The basic seismic parameters developed in the early stages may be updated at this time if significant changes have occurred in the understanding of potential motions.

A target response spectrum should be defined for the materials that underlie the embankment and upper foundation. This will typically be a stiff soil layer or bedrock, and typically corresponds to the bottom of a two-dimensional finite element or finite difference grid. In special cases it may be necessary to define the response spectrum at a surface location and then deconvolve the motions down to the appropriate depth below the ground surface.

At least four or five ground motion records are generally needed for advanced deformation analyses. A relatively large suite of records is required due to the range of deformation predictions that may occur even for a carefully selected set of motions. Since only 4 or 5 records are being used, the intent of the study is not to define the full range of potential displacements but to determine the average, expected response for the specified level of earthquake loading.

Three ground motion components should be provided for each record: two horizontal and one vertical. While vertical motions are often considered to have a modest effect on deformation predictions, advances in developing appropriate and consistent motions and the ease with which they can be included in many sophisticated analyses warrants their routine use in deformation analyses.

The suite of records should be obtained from different source earthquakes to reduce unintended bias in the record selection. The following criteria may be considered in the selection of earthquake record:

- Records to be used for preparation of site specific histories should originate from a seismic event similar to the target design earthquake (e.g., magnitude, fault distance, and focal depth).

- The site condition for each record should reasonably correspond to the site condition for the target response spectrum. For example, it may be appropriate to use a record from a shallow, stiff soil site to represent soft rock conditions, but not a deep soil record.

- The shape of the response spectrum for each record should reasonably match the target response spectrum over the frequency range of interest. This frequency range may be rather large and will typically include short frequencies (long periods).

- Scaling factors may be applied to the record to provide a best fit to the response spectrum over the period range of interest (see Fig. 33). Alternatively, spectral matching programs such as RSPMatch can be used to more closely follow the response spectrum over a wide range of frequencies. Spectral matching techniques should be carefully applied to preserve as much of the original character of the earthquake record as possible (e.g., relative magnitude and duration of velocity peaks).

- Scaling factors are traditionally limited to values between 0.5 and 2.0, although values outside of this range may be permitted in some cases (Watson-Lamprey and Abrahamson, 2006).

- Additional criteria can be useful in defining an appropriate suite of ground motions, such as Arias Intensity or significant duration. Attenuation relationships are available for these parameters allowing their inclusion in deterministic or probabilistic hazard estimates (e.g., Watson-Lamprey and Abrahamson, 2006; Travasarou, et al., 2003; Kempton and Stewart, 2006].

Original earthquake records can be obtained from a number of online sources, including the COSMOS and PEER websites. Synthetic accelerograms should be considered when the design earthquake is not well-represented by the database of available records.

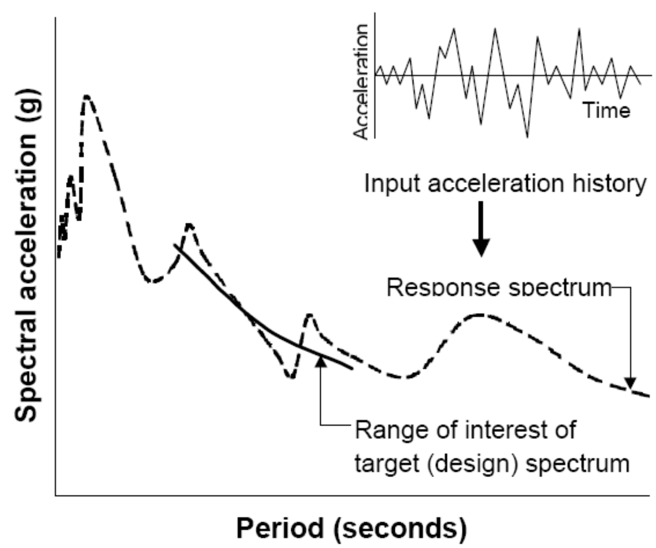


Fig. 33. Scaling of the input time history for best fit with target response spectrum over period range of interest (Perlea, 1999).

Determination of Soil Properties

Field tests are generally preferred when liquefiable soils are present. A combination of standard penetration tests (SPT) and cone penetration tests (CPT) often gives the best and most reliable assessment. Cyclic laboratory tests can also be useful when high quality undisturbed samples are available (Castro et al. 2003). Many analyses can be improved if combinations of empirical and laboratory-based approaches are used, as long as suitable samples can be obtained.

Standard Penetration Test: The normalized blowcount, $N_{1,60}$, from the SPT is a useful parameter for the evaluation of both triggering of liquefaction and the post-liquefaction residual strength of cohesionless materials. It is the Corps policy to require calibration of the equipment and operator at each investigation site. It is also noted that some conservatism is necessary in the selection of the representative $N_{1,60}$ parameter for evaluating liquefaction triggering when the soil units are defined by relatively large zones. The early developers of empirical triggering charts for liquefaction attempted to identify looser sublayers that may have initiated liquefaction when the available data allowed a detailed interpretation. Characterizing large zones with average or median blowcounts may underpredict their tendency to liquefy. This was also suggested by a limited evaluation performed by Popescu et al. (1998). The 33rd-percentile $N_{1,60}$ estimate has generally been used for evaluation of liquefaction triggering resistance, while 33rd to 50th percentile estimates are used for residual strength.

The presence of gravels can significantly impact the reliability of the SPT measurements. Where gravels are suspected, the hammer blows should be reported for every 1 inch of penetration rather than the standard 6 inches. This may allow evaluation of the sand matrix between isolated gravel particles. This approach may be less useful in soils with higher gravel contents since these gravels can still affect the penetration through limited zones of the sand matrix. Pervasive gravels may require other techniques, such as the Becker Penetration Test (BPT).

<u>*Cone Penetration Test (CPT)*</u>: The CPT is useful in evaluating soil stratigraphy including estimates of soil type and layering. Although its relative cost makes it an attractive investigation tool, it is typically used in conjunction with SPT tests to allow soil samples to be obtained for confirmation of soil type. Developing a site-specific correlation between CPT and SPT is often useful to confirm interpretations in sandy materials and to determine the extent of liquefiable zones. The CPT can also be used in the initial investigation of the dam and foundation to determine the scope of the required investigation program and to identify locations where SPT, field vane tests, and undisturbed samples should be obtained.

<u>*Becker Hammer Penetration Test (BPT)*</u>: The BPT can be useful when gravels or gravelly sands are investigated. The rate of penetration is used to estimate the equivalent SPT blowcount in materials that are too coarse to be reliably tested by the SPT or CPT. At Success Dam, BPT and large penetration tests (LPT) were used in the alluvial and embankment shell materials (Serafini et al. 2008). BPT and LPT are not directly used in liquefaction assessment, but only through correlations with SPT values based on published relationships and, preferably, site specific correlations.

<u>*Field Vane Test (FVT)*</u>: The FVT is used in clays and other fine-grained materials for estimating the undrained strength. Residual shear strength at large shear strain may also be obtained which can be useful in estimating the post-earthquake strength of soft clay soils. The FVT was used at Tuttle Creek Dam, in conjunction with laboratory tests on good quality relatively undisturbed samples, for characterizing the dynamic behavior of low plasticity cohesive soils in foundation. Fig. 34 presents the results of these field tests.

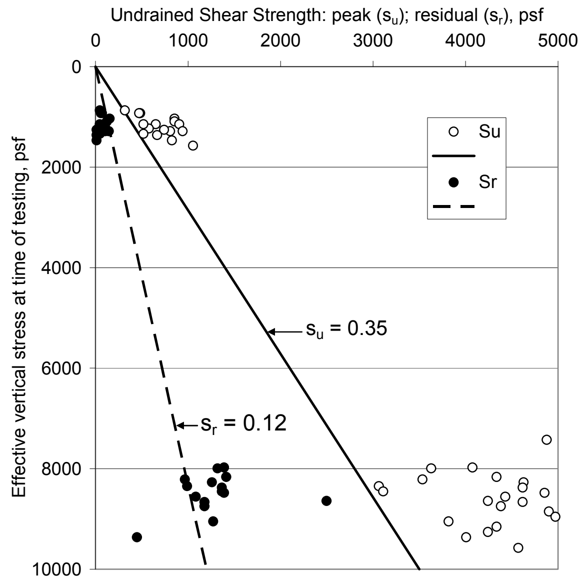


Fig. 34. Results of FVT at Tuttle Creek Dam: open circles – peak strength; solid circles – residual strength; upper dots – soundings in free field, near embankment; lower dots – soundings through mid-slopes (after Castro et al. 2003).

<u>*Shear Wave Velocity (Vs)*</u>: The small strain shear wave velocity, V_s, is directly related to the maximum shear modulus of the soil, G_{max}. This parameter is useful in constraining the expected stress-strain response of the soil. It can also be used to assist in liquefaction evaluation through correlations between CRR and V_s (Youd et al., 2001; Liu and Mitchell, 2006). The V_s data was used at Success Dam to support the evaluation of representative $N_{1,60}$ developed from SPT, LPT, and BPT. A comparison between V_{s1} and $N_{1,60}$ was developed for this project as shown on Fig. 35.

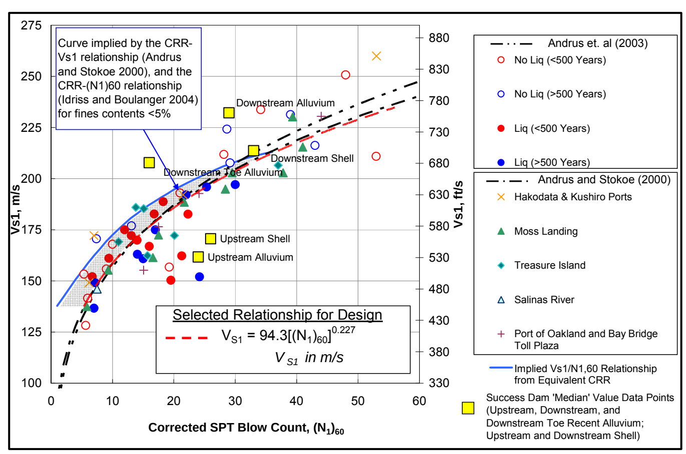


Fig. 35. Conversion of the normalized shear wave velocity into an equivalent $(N_1)_{60}$ for Success Dam (Serafini et al. 2008).

Among the various geophysical methods for measurement of shear wave velocity, the cross-hole method has been generally preferred by the Corps of Engineers. However, the presence of stronger materials above and/or below a relatively thin weak layer may hide its critical properties when using this method. This was the case for a 5 foot thick loose alluvium layer in the foundation of the Isabella Dam. Suspension logging (also known as P-S logging or Oyo logging) was selected in this case and furnished more detailed data for relatively thin layers than the cross-hole soundings (Fig. 36).

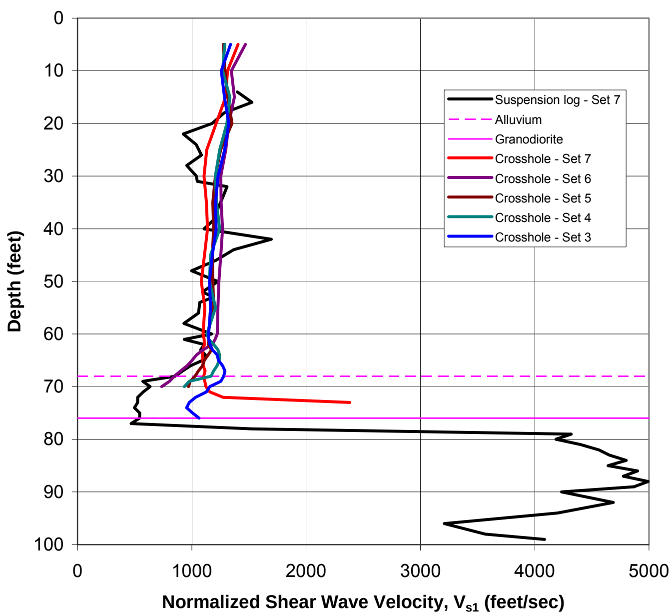


Fig. 36. Geophysical investigation for V_{s1} at Isabella Dam, California.

Laboratory Cyclic Tests: Laboratory tests may be needed for the evaluation of the cyclic resistance or post-liquefaction response of some soils, such as soils with low to modest plasticity. Such materials are frequently encountered in alluvial deposits in Midwest United States. The top 15-20 feet of the foundation soil at Tuttle Creek Dam was such a material (clay content 10-20%, fines content 68-100%, liquid limit 23-32, plasticity index 1-12). GEI Consultants, Inc. assisted in undisturbed sampling and testing of these samples in the cyclic triaxial device (Castro et al. 2003).

One of the most important findings of the GEI study was that the undrained strength at the end of cyclic loading remained high if the cyclic strain was small (Fig. 37). A tendency for the undrained strength to decrease was observed within the limits of laboratory tests, trending towards the large strain strengths from the field vane tests.

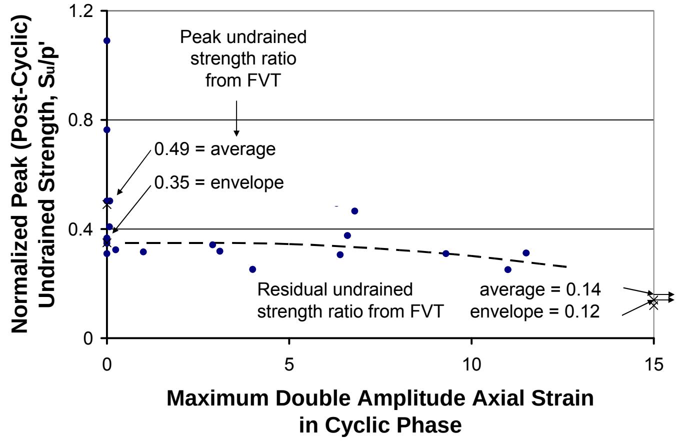


Fig. 37. Peak (post-cyclic) undrained shear strength ratio versus maximum axial strain developed at the end of cyclic loading: solid circles are used to plot the laboratory test results; field vane test data are shown with x's (based on Castro et al. 2003).

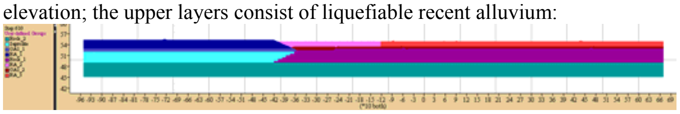
Static Effective Stress Condition

For many seismic deformation analyses, the purpose of the static analysis is to obtain a reasonable distribution of the initial mechanical and pore water stresses. Since there is no single answer to this problem, but rather an infinite set of possible solutions that satisfy the imposed boundary conditions, overly detailed modeling for the static analysis may not be warranted. A careful modeling of the initial dam construction, reservoir filling, and remediation construction may be advantageous when there are significant stiffness contrasts between zones or other factors that may impact the stress distribution. In some cases it may be useful to evaluate more than one initial static stress distribution to estimate its effect on the resulting deformations.

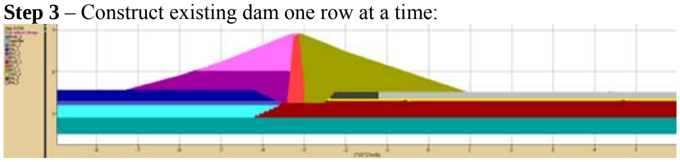
Relatively simple constitutive models may often be used for the static analysis phase. The dependence of stiffness on effective stress level is an important consideration in many cases. Plastic behavior should be included in the model to prevent elements from developing unrealistic stress states. It is often desirable to plot stress contours, including vertical, lateral, and shear stress, to look for unusual or unexpected stress concentrations or distributions. A plot of the coefficient of lateral earth pressure K_0 is generally useful in evaluating the stress state.

A rather detailed modeling of the initial construction and future remediation process was performed as part of the FLAC analysis of Success Dam. Fig. 38 presents the steps followed for estimating the initial stress state before simulation of the seismic loading of the preferred remediation alternative. The placement and excavation of the embankment was performed in a number of substeps to allow the model to establish equilibrium between the addition or removal of each soil

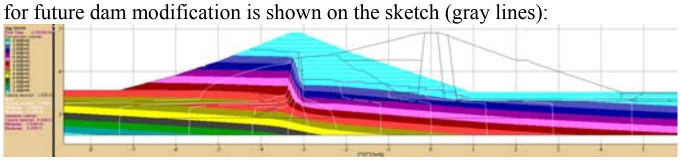
Step 1 – Bring original foundation to equilibrium with the original water table elevation; the upper layers consist of liquefiable recent alluvium:



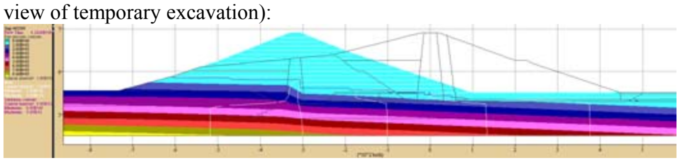
Step 2 – Excavate core trench (down to water level).
Step 3 – Construct existing dam one row at a time:



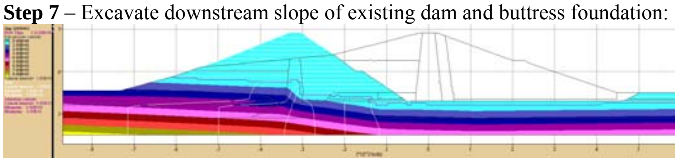
Step 4 – Steady-state seepage analysis at gross pool (spillway crest); the plan for future dam modification is shown on the sketch (gray lines):



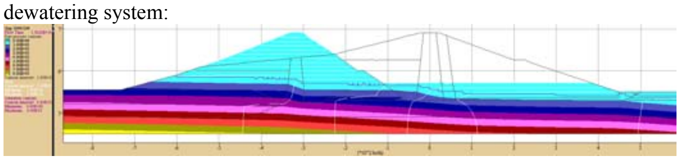
Step 5 – Steady-state seepage analysis for drawdown to conservation pool (in view of temporary excavation):



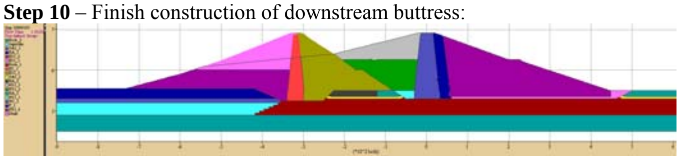
Step 6 – Steady-state seepage analysis with dewatering wells (in view of temporary excavation.
Step 7 – Excavate downstream slope of existing dam and buttress foundation:



Step 8 – Construct lower portion of the downstream buttress, high enough to allow safe removal of the dewatering wells.
Step 9 – Perform steady-state seepage analysis for the condition without dewatering system:



Step 10 – Finish construction of downstream buttress:



Step 11 – Degrade crest of existing dam;
Step 12 – Steady-state seepage analysis at projected gross pool:

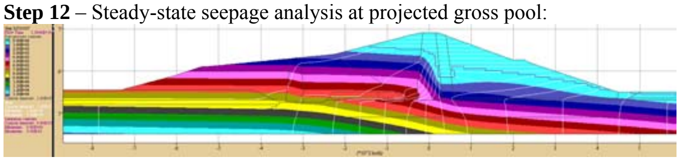


Fig. 38. Steps used to simulate construction of Success Dam including preferred remediation.

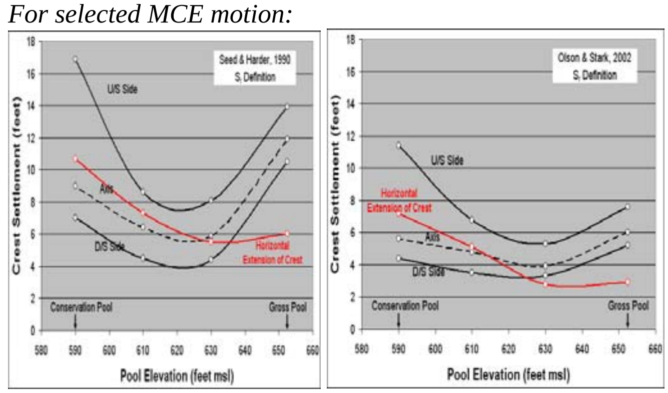
layer. The Mohr-Coulomb model with stress-dependent stiffness properties was used to model the soil in this series of analyses.

<u>Coincident Pool Level</u>

The elevation of the reservoir and the initial pore pressure distribution within the dam can have a significant impact on the predicted deformation. The Corps of Engineers manual EM 1110-2-2100 defines the coincident pool as follows: *"Coincident pool represents the water elevation that should be used for combination with seismic events. It is the elevation that the water is expected to be at or below for half of the time during each year."* Since most of the Corps dams were built primarily for flood control, the coincident pool is generally rather low. A low pool provides minimal buttressing from the reservoir on the upstream slope, but also results in a significant height of initial freeboard and high initial effective stresses. Evaluating just the low pool condition for a flood control dam may not provide the best assessment of risk, and a study of the response versus reservoir level may be required.

For Success Dam, the low (conservation) pool was initially assumed to be the most critical level for seismic displacements in the upstream direction. This assumption was later evaluated through a series sensitivity analysis with reservoir pools at various elevations as shown in Fig. 39 (Ruthford et al. 2008).

For selected MCE motion:



For selected OBE motion:

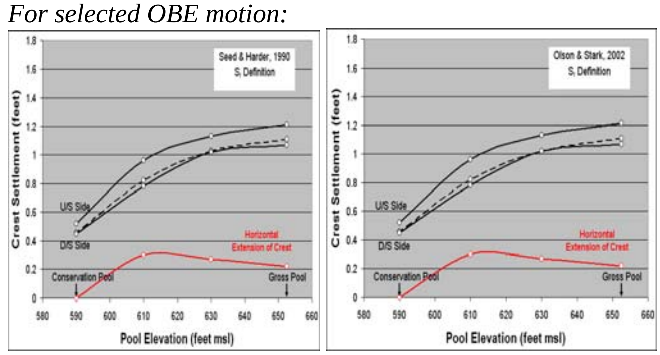


Fig. 39. Predicted crest settlement for various residual strength assumptions and reservoir pool elevations (Ruthford et al. 2008).

That study found that the highest assumed pool may be as critical as the low pool. The changes in the initial effective and shear stresses caused by the various reservoir levels result in changes in the liquefaction response of the critical zones. Because the higher pool is more critical from the dam safety perspective, the gross pool at spillway crest was primarily used for subsequent analyses. The lower, coincident pool was used for verification at the end of the analysis.

Constitutive Models

A wide range of constitutive models can be successfully used in advanced deformation analyses. A number of considerations should be made when selecting a constitutive model for each soil zone in the model:

- The formulation of the constitutive model should adequately address the key features of the anticipated soil behavior. These may include the relationship between shear stiffness and strain, stress-level dependence, generation of pore pressures, and/or strain softening.

- The constitutive model should have a sound theoretical basis.

- It can reasonably model both monotonic and cyclic behavior as observed in laboratory tests. Direct comparisons should be made between numerical simulations and laboratory test data (site specific and/or relevant published information). For liquefiable soils, these comparisons may include relationships for triggering resistance, effect of initial static shear stress on liquefaction resistance, and influence of effective overburden stress.

- For most cases involving liquefaction, the use of the empirical relationships for residual strength based on case histories should be directly considered in the choice of constitutive models and their use in the analysis.

- The selection of input parameters should be reasonably transparent, particularly in cases where direct calibration to laboratory data is not possible.

- The constitutive model has been successfully used to simulate observed deformations from case histories.

The project documentation related to the choice of constitutive models should address these topics, particularly for the soil zones that are critical to the response of the structure. The behavior of each model within the structure should be verified by evaluating predicted element output (e.g., see Fig. 19, Fig. 23, and Fig. 24). The goal of the documentation should be to provide present and future reviewers a transparent understanding of the advantages and limitations of the constitutive model for the range of loading conditions to be experienced by the dam.

The Corps has used a number of constitutive models for recent analyses of dams. The UBCSAND model, as described above, has been successfully used for both final evaluation of existing structures and for remediation designs. Fig. 40 to Fig. 43 show selected examples of element calibrations for both the UBCSAND and UBCTOT constitutive models. The amount of similarity between a numerical simulation and laboratory data often varies even for a single set of test data. However, the general model behavior should be demonstrated and adequately confirmed through such comparisons.

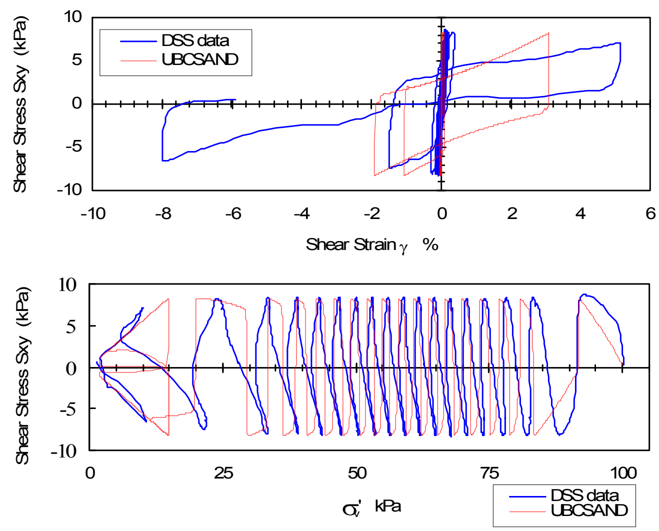


Fig. 40. Example comparison between stress-strain and stress-path behavior of UBCSAND simulation versus laboratory test data.

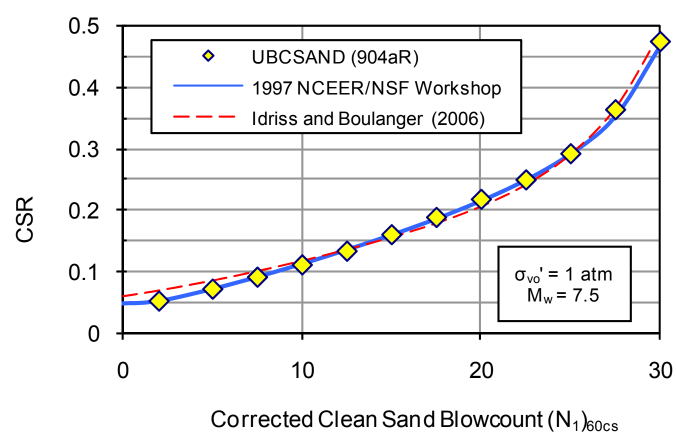


Fig. 41. Comparison between predicted CSR from UBCSAND versus semi-empirical relationships expressed in terms of SPT blowcount.

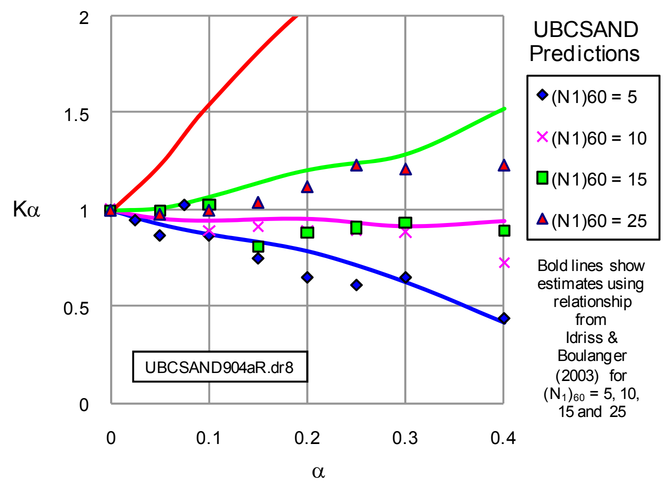
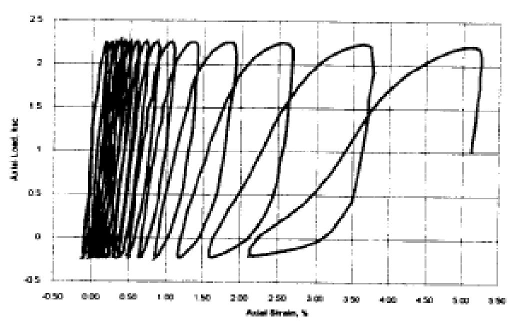


Fig. 42. Comparison between predicted effect of initial static shear stress ($\alpha = \tau_s / \sigma_v'$) from UBCSAND (version 904aR) versus K_α relationship proposed by Idriss and Boulanger (2003).

a) Triaxial test results:



b) UBCTOT simulation:

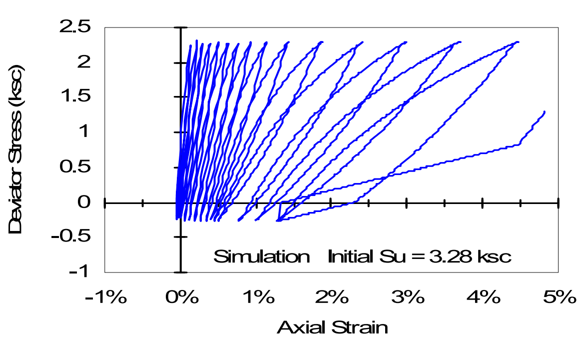


Fig. 43. Comparison between triaxial test results and modified UBCTOT simulation for Tuttle Creek Dam (Stark Consultants, 2007).

<u>Dynamic Response Analysis</u>

There are a number of computer programs and corresponding constitutive models that are capable of modeling the dynamic behavior of embankment dams, including cases where potentially liquefiable materials are present. Many of these programs can be used to successfully evaluate a structure if they are properly applied and the input and results are carefully reviewed. The Corps of Engineers currently gives preference to FLAC analyses. The adoption of this program considered its commercial availability, its wide use within the geotechnical profession, and the potential for applying user-defined constitutive models.

While the dynamic analyses of embankment dams primarily focus on the magnitude and pattern of the displacements, several other factors should be reviewed. For example, the predicted extent of liquefaction, acceleration histories at key locations (e.g., crest, slopes, and free-field), and predicted element behavior at important locations (e.g., stress and pore pressure histories, shear stress versus shear strain, and stress path). The element behavior of selected elements should be compared to the anticipated behavior, and estimates such as peak crest acceleration and predicted crest settlement should be compared to empirical relationships for a general check of reasonableness (e.g., US Army Corps of Engineers, 2000; Swaisgood, 2003).

A well-planned parametric study should be performed to evaluate both the stability of the numerical predictions as well as the fragility of the structure. The anticipated displacements of some structures can increase dramatically with rather small changes in loading or material properties. This is particularly true for structures with liquefiable or sensitive soils.

Evaluating the relative change in response between a remediated and non-remediated section is an important consideration in developing the remediation plan. The final analysis tools, including both computer program and constitutive models, should be applied to both the original and remediated sections to facilitate this comparison. Fig. 44 to Fig. 46 show a comparison of predictions for Success Dam.

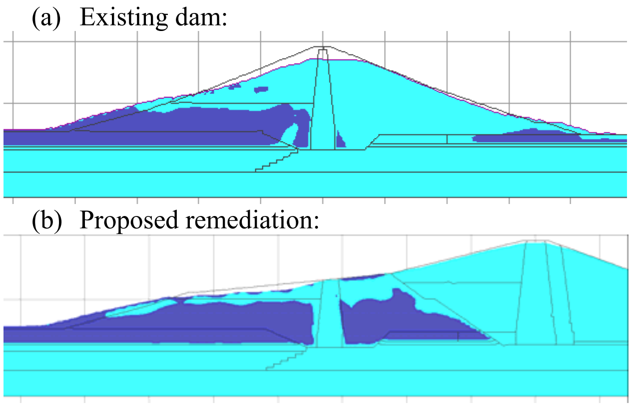


Fig. 44. Predicted zones of liquefaction for MCE at Success Dam.

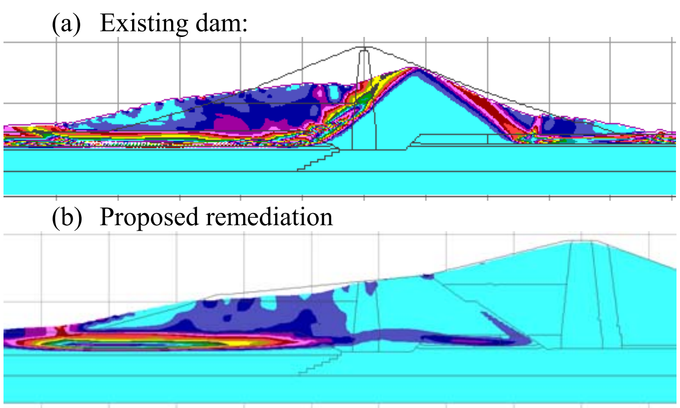


Fig. 45. Predicted contours of maximum shear strain for MCE at Success Dam (contour interval γ = 20%).

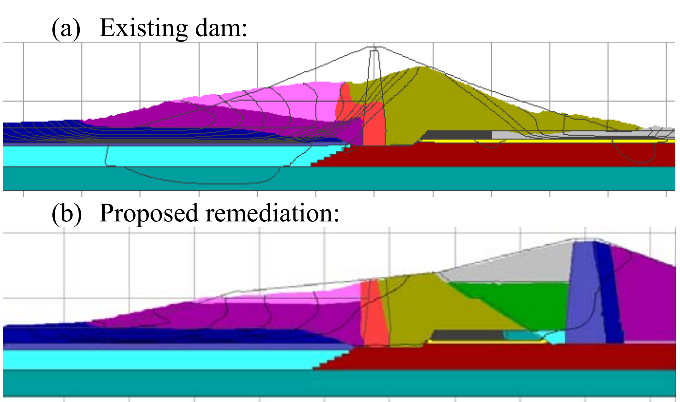


Fig. 46. Predicted deformations and contours of horizontal displacement for MCE at Success Dam (displacement contour = 10 feet).

Interpretation of Results

Allowable deformations: The Corps of Engineers regulations (ER 1110-2-1806 "Earthquake design and evaluation for civil works projects") allow two different levels of acceptable damage with respect to the level of seismic loading:

- Under the Operating Basis Earthquake (OBE) the project should function with little or no damage, and without interruption of function.

- Under the Maximum Credible Earthquake (MCE), the project should perform without catastrophic failure, such as an uncontrolled release of the reservoir. Severe damage or economic loss may be tolerated. For critical features such as dams, the Maximum Design Earthquake (MDE) is the same as the MCE.

These statements provide general guidance on interpreting the results of a deformation analysis. It is not possible to specify a maximum displacement criterion that applies to all projects for a variety of reasons. Many factors can vary significantly between dams and can influence the magnitude of allowable deformation. These factors include the following items: site conditions; earthquake loading; dam design features such as slopes, location and design of filters; soil properties; quality of site characterization; regularity of foundation and steepness of abutments; narrowness of canyon; available freeboard; downstream consequences; sophistication and applicability of analysis procedure; results and quality of parametric study; and whether the dam is an existing, remediated, or new embankment. Allowable deformations should consider not only the anticipated residual freeboard but the potential for erosion through transverse cracks that may develop during the earthquake. For these reasons, it is necessary to develop and justify deformation criteria on a case by case basis.

The list of observations and general descriptions compiled by Swaisgood (2003) and Pells and Fell (2003) may be helpful in developing allowable displacement criteria. Swaisgood suggests a general damage rating system as summarized in Table 1. This rating system, however, is qualitative and may not directly correspond to allowable displacements at any particular dam.

Table 1. Damage versus crest settlement from historic earthquake response (Swaisgood, 2003).

Crest Settlement (% of DH + AT)	*Damage Rating*
> 0.5%	Serious
0.1% to 1%	Moderate
0.02% to 0.5%	Minor
< 0.1%	None

Note: DH = dam height, AT = alluvium depth in foundation

Pells and Fell (2003) provide additional information on observed dam behavior. Post-earthquake observations of 305 dams were compiled and evaluated. Relative crest settlements of 0.2% to 0.5% of the dam height were found to be associated with a high likelihood of transverse cracking. Transverse cracking was found after earthquakes with magnitudes less than 6 to 6.5, and at PGA values as low as 0.1 to 0.15g. Relative crest settlements in the range of 1.5% to 5% were described as causing severe damage. When applying the observations of Swaisgood (2003) and Pells and Fell (2003), it is important to remember that the observations strictly apply to dams that do not experience liquefaction in the embankment or foundation.

Another factor that must be considered is the range of deformations predicted by the analyses. A suite of well-chosen earthquake records representing the same design event may still show a range in the predicted crest displacements that exceed a factor of 3 or more. While the goal of these analyses is to predict the likely or median response to the specified loading, the predicted range of loading should be considered. A wide scatter in predicted displacements should be carefully reviewed and understood.

Advanced deformation analyses that are performed well and with sufficient site characterization are often considered to give displacement predictions that are within a factor of about 2 of the true expected displacement. This number is not objectively supported, but is an informal assessment of results from case history analyses and general predictions. However, not all analyses are equal and the potential accuracy should consider the appropriateness of the constitutive models, the level of detail achieved in understanding the site and material properties, and the amount of care exercised in performing the analyses and parametric studies.

Gilles Bureau (1997) stated his assessment regarding the validity of deformation analysis results for use in evaluating acceptable freeboard. In his opinion, *"non-linear deformations obtained from well-verified computer programs and dependable input data should be acceptable up to calculated ratios of crest settlement to dam height of about ten percent. Use of larger relative settlements as a basis to define a safe freeboard would be speculative. The above ratio applies to wide dams, built on a regular foundation and gently sloping abutments. Engineers must consider factors that may reduce such a ratio, such as dam zoning, percent compaction, wet or dry condition from optimum moisture content, shape of canyon and abutments, and deformability and plasticity of the embankment materials."*

As an example, Fig. 47 presents the allowable displacement criteria accepted by the Corps of Engineers for the evaluation of the Tuttle Creek Dam before improvement and for the design of remediation.

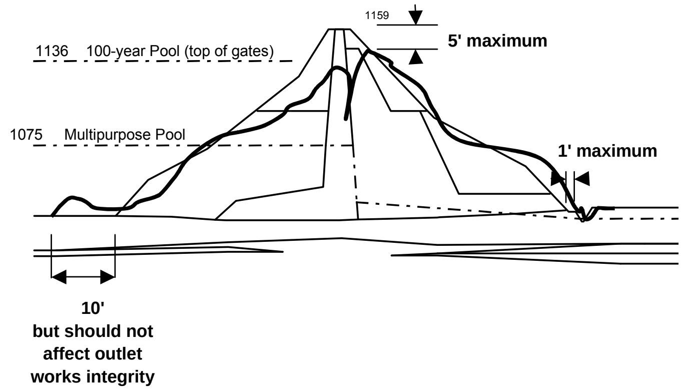


Fig. 47. Displacements considered acceptable under the MCE at Tuttle Creek Dam (not to scale).

Justifications for these criteria for Tuttle Creek Dam included the following:

- The acceptable limit of 10 feet (3 m) for horizontal displacement of the upstream toe considered the danger of fracturing the natural impervious blanket immediately upstream of dam. Damaging this blanket could lead to increased underseepage beneath the dam.

- Five feet (1.5 m), or about 3.6% of the dam height, was considered an acceptable loss of freeboard due to an MCE event. Designed primarily for flood control, the dam has a large freeboard of 84 feet (25.6 m) at the coincident pool, and 23 feet (7 m) at the 100-year pool.

- Only 1 foot (0.3 m) of horizontal displacement was considered tolerable at the downstream toe due to potential damage to the relief well system installed immediately below the toe of the dam. In the absence of a positive cutoff, the relief wells are a vital feature of the dam.

The conditions were different at Success Dam and so were the allowable deformations. The reliefs wells, although efficient in relieving water pressures at the downstream toe, are not vital features since the existing cutoff is considered effective. Although Success Dam is primarily a flood control dam, the computed deformations for the existing dam under the MCE were so large that they well exceeded any reasonable displacement criterion. The preferred remediation alternative also considers large deformation at the upstream toe, up to about 45 feet (14 m) at the upstream toe, but these were considered acceptable for dam stability since they occur in a region that does not impact the safety of the dam. However, additional remediation is required in the vicinity of the embedded outlet tower and intake structure to reduce the displacements at these locations. The predicted displacements of the crest, 4.0 feet (1.2 m) vertical, and of the downstream toe, 2.9 feet (0.9 m) horizontal, were also judged acceptable.

<u>Risk analysis</u>: Seismic deformation analysis may be used to help quantify the risk of some failure scenarios, such as embankment overtopping after an earthquake or piping failure due to erosion through cracks.

One example of deformation analyses being used to support of a risk analysis is the recent evaluation performed for the existing section of Success Dam. Sixty deformation analyses were performed on the critical cross section for various loading conditions. A range of earthquakes were considered based on the probabilistic seismic hazard assessment, and these were grouped into three magnitude ranges: M > 7; M = 6.5 - 7; and M < 6.5. One representative acceleration history was selected for each magnitude range based on previous analyses, and these records were scaled to various values of PGA. Four different reservoir elevations were considered: spillway crest (highest), conservation pool (lowest), and two intermediate levels. Crest settlement was used to quantify the anticipated extent of damage, and plots of predicted settlement versus PGA are shown in Fig. 48.

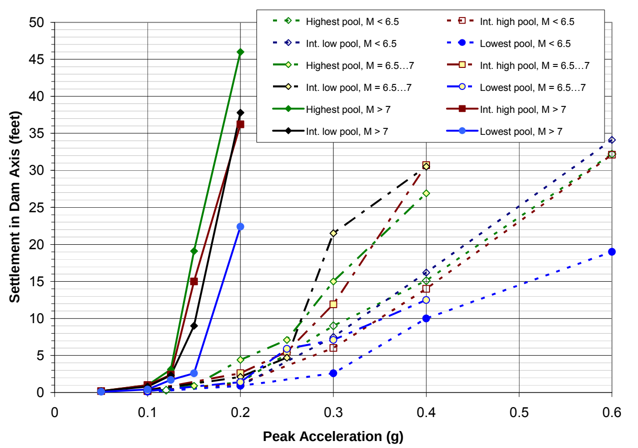


Fig. 48. Success Dam Risk Assessment: Downward vertical displacement of crest as a function of PGA.

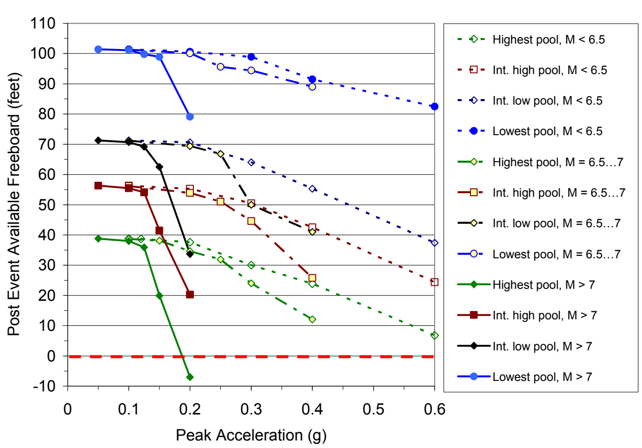


Fig. 49. Success Dam Risk Analysis: Available freeboard at the dam axis after seismic deformation as a function of the peak ground acceleration.

The crest settlement data was also plotted in terms of residual freeboard, as shown on Fig. 49. Immediate overtopping of the dam after an earthquake was found to be a rare event.

One of the points plotted on Fig. 49 shows a negative freeboard. This indicated the final predicted crest elevation is less than the pool elevation, and overtopping of the crest will occur. However, for this case it did not mean that the dam itself would immediately overtop. The predicted deformed shape for this case is shown in Fig. 50. Even though the core drops below the pool level, there is still the potential for a thick remnant of the downstream shell to retain the pool. This remnant was estimated from the deformation analysis as having a width of about 95 feet (29 m). If the actual deformations had a similar pattern but were 50% larger than computed, there is still an estimated 60 feet (18 m) of shell material to prevent immediate overtopping. The final condition of this shell material and its ability to safely retain the reservoir is a second consideration.

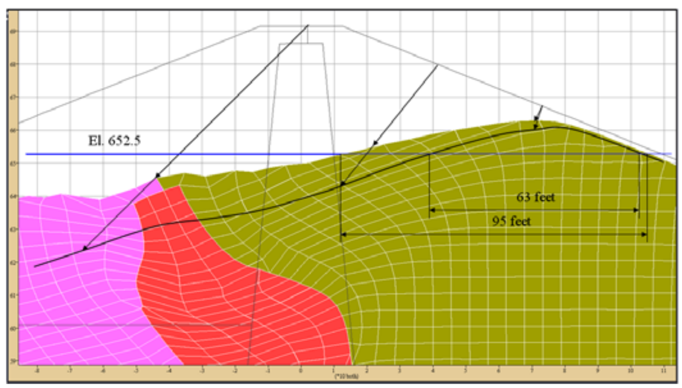


Fig. 50. Worst case deformations predicted for risk analysis of Success Dam. Water surface is at elevation 652.5.

The predicted drop in core, as shown on Fig. 50, results in the direct contact of the reservoir with the relatively pervious shell. While not an ideal situation, it is a significant consideration in a risk analysis since the shell could delay or possibly eliminate a breach of the dam.

CONCLUSIONS

Guidance for the Corps of Engineers requires the use of seismic deformation analysis in the evaluation of existing dams for seismic loads and in the validation of remediation design for seismically deficient embankment dams. The Corps uses a phased approach for evaluating the seismic safety. Simple tools, such as screening methods, Newmark analyses, or post-earthquake limit equilibrium evaluations, are useful for either preliminary assessment or for structures with relatively little seismic concern. Since these simple analyses can be viewed as providing a displacement index, they should be applied carefully and with explicit conservatism.

For dams with significant seismic loads, problem soils, high risk, or those where simple analyses have identified seismic concerns, a more advanced deformation analysis is generally required. The approach used by the Corps to perform these advanced analyses has continued to evolve over time. The primary objective of this paper is to describe the current practice of the Corps and to discuss selected considerations in performing these analyses.

A major challenge in performing an advanced deformation analysis is the selection of reliable constitutive models and a suitable computer code. The Corps has recently used several computer codes and constitutive models to evaluate the behavior of embankment dams that include potentially liquefiable materials. Based on this experience, the Corps currently uses the computer program FLAC as the primary tool in both evaluating the non-remediated behavior of embankment dams and in designing any required remediation. The selection of this program considered its commercial availability, its wide use within the geotechnical profession, and the potential for applying user-defined constitutive models.

FLAC, in conjunction with a modified version of the UBCSAND constitutive model, has been instrumental in determining the necessity of seismic retrofit at Corps' embankment dams and in selecting effective remediation alternatives. UBCSAND has been successfully used in recent projects and is the current choice of the Corps for modeling liquefiable materials. One important aspect of constitutive model selection is the proper documentation and evaluation of the model behavior under the anticipated range of stress and loading conditions to be experienced by the dam.

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